MITIGATION OF DISASTERS IN HEALTH FACILITIES

ENGINEERING ISSUES

VOLUME 4

Pan American Health Organization
Regional Office of the
World Health Organization

MITIGATION OF DISASTERS IN HEALTH FACILITIES

EVALUATION AND REDUCTION OF PHYSICAL AND FUNCTIONAL VULNERABILITY

VOLUME IV: ENGINEERING ISSUES

PAN AMERICAN HEALTH ORGANIZATION
Regional Office of the
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PREFACE

The series of documents entitled *Mitigation of Disasters in Health Facilities: Evaluation and Reduction of Physical and Functional Vulnerability* has been prepared by the Pan American Health Organization for national, provincial, or municipal authorities (Volume I: General Issues); owners of buildings, administrators, staff members, and other personnel connected with health installations (Volume II: Administrative Issues); designers, architects, builders, and educators (Volume III: Architectural Issues); and for design engineers, planners, builders, and educators (Volume IV: Engineering Issues).

The purpose of the series is to inform the people involved in the planning, operation, management, and design of health services concerning possible effects of natural disasters on health installations. The idea is to provide a useful tool that makes it possible to incorporate risk mitigation procedures both in the inspection of existing installations and in the design and construction of new buildings and services.

Each volume in the series deals with specific subjects related to the potential problems that can arise when a disaster occurs and, also, discusses the measures that should be taken to mitigate risk, placing special emphasis on the necessary requirements to ensure that installations can continue functioning during and immediately after a sudden impact disaster.

Although health installations can be affected by a broad spectrum of natural phenomena such as earthquakes, hurricanes, landslides, volcanic eruptions, floods, etc., as well as by man-made disasters, such as fires, explosions, gas leaks, and others, the series emphasizes the seismic problem, given that it is the natural phenomenon that has most affected health installations in the world and since, if its direct and indirect effects can be reduced, the risk posed by other phenomena, whose impact is normally less than that which earthquakes can cause, will also be lowered.

The manuals for architects and engineers address professionals familiar with architectural design and with structural analysis and design, respectively. Their approach is to raise concern about traditional techniques and to contribute proposals that are not usually to be found in the standard, specialized reference books.

The Pan American Health Organization/World Health Organization has chosen to promote the preparation and publication of this series as a contribution to the goals of the International Decade for Natural Disaster Reduction (IDNDR).

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On cover: *The earthquake that jolted Mexico City on 19 September 1985 was the strongest one recorded in Latin America in the last hundred years. This earthquake killed and wounded thousands and caused severe structural damage. Health sector institutions also suffered tremendous blows, among them the General Hospital of the National Medical Center of the Mexican Social Security Institute, shown in the photo.*

Photography: Julio Vizcarra/PAHO

INTRODUCTION

The planning, design, and construction of hospitals in areas prone to natural hazards poses many challenges to the various professionals involved not only because of the hospitals' importance in normal city life but also because of the importance they assume if the victims of a disaster must be cared for. Given the significance of hospitals for the recovery of an affected community, for example in the event of a strong earthquake, careful consideration should be given to a wide range of aspects, from planning disaster response to installing equipment and various non-structural equipment and elements, in addition to the requirements of structural resistance and safety.

Despite the foregoing, many hospitals have suffered serious damage or have undergone functional or structural collapse as a result of disasters, in particular in the case of intense earthquakes, thus depriving the respective communities of adequate care for victims.

It bears noting that many of the damaged hospitals have been designed in accordance with standards of seismic-resistant construction. This suggests that the structural design of hospitals should be carried out with much greater care than in the case of more conventional designs and that it may not be enough to simply plan the structural design for forces greater than those used in calculations for residential or office buildings. The structural design should include decisions on safety, resistance, and ductility, not only with regard to the purely physical aspects that earthquakes entail but also with regard to social, economic, and human criteria that bear on the planning of the hospital.

Chapter 1 of this manual examines briefly the concepts relating to the characteristics of disasters, in particular seismic hazard. Chapter 2 deals with general considerations relative to the structural analysis and design of hospital buildings. It reviews the criteria governing design, such as probability of occurrence, energy absorption, acceptable risk, functional safety, and mechanisms to isolate buildings and control vibrations.

Chapter 3 analyzes the factors that make hospital buildings vulnerable. It discusses the problems of physical vulnerability, such as architectural-structural configuration. It also analyzes the problems of functional vulnerability that could lead to the collapse of hospital service after an intense event, such as potential damage to facilities, equipment, and non-structural elements.

One of the most delicate aspects in the consideration of the seismic outlook in countries of Latin America and the Caribbean is the performance of existing hospitals that have been constructed without seismic-resistant design. Chapter 4 discusses the techniques for evaluating the vulnerability of this type of building and the methods of repair or retrofitting.

Finally, Chapter 5 deals with professional training in the specific area of the engineering design of hospital facilities. It suggests curriculum adaptations and the programming of continuing education courses to promote these nontraditional aspects in the training of design and construction engineers.

This document is designed for engineers with previous experience in structural analysis and design. It illustrates the fundamental concepts concerning the performance of buildings under seismic stresses and techniques for evaluating and rehabilitating structures. However, this document does not replace the books containing detailed theoretical information on seismic and structural engineering, books to which the text very often refers.

CHAPTER 1

CHARACTERISTICS OF DISASTERS

TYPES OF DISASTER

A disaster can be defined as an event that occurs in most cases suddenly and unexpectedly and causes severe disturbances in the form of loss of life and health among the population, the destruction or loss of a community's assets, and severe damage to the environment. The result is the disruption of normal life, causing misfortune, helplessness, and suffering among the people, effects on the socioeconomic structure of a region or country, and changes in the environment, thus dictating the need for immediate assistance and intervention.

Disasters can be caused by a natural phenomenon, can be man-made, or can result from a technical failure in industrial or military systems.

Some natural disasters represent hazards that cannot be neutralized, inasmuch as they can hardly be forestalled, although in some cases they can be controlled to a degree. Earthquakes, volcanic eruptions, tsunamis (tidal waves), and hurricanes are examples of hazards that cannot yet be prevented in practice, while floods, droughts, and landslides can be controlled or mitigated through public works involving drainage and soil stabilization.

The following is an extensive list of the natural phenomena that can cause disasters or calamities:

- Earthquakes
- Tsunamis (tidal waves)
- Volcanic eruptions
- Hurricanes (storms, high winds)
- Floods (slow, rapid)
- Mass movements (landslides, collapses, flows)
- Droughts (desertification)
- Epidemics (biological)
- Pests

These are the primary phenomena, but others can result from them, such as avalanches or mudslides and the precipitations or flows of pyroclastic material that are directly associated with volcanoes, as well as other phenomena that can be grouped together as equivalents, such as tornados, tropical cyclones, or hurricanes. Most of these phenomena occur cataclysmically, that is to say, suddenly, and do not affect a very large area. However, desertification and droughts take place over a long period and affect extensive areas in an almost irreversible manner.

Manmade disasters can be caused deliberately or can be due to a technical defect that triggers a string of failures causing a major disaster.

Among other manmade disasters, the following can be mentioned:

- Wars (terrorism)
- Explosions
- Fires
- Accidents
- Deforestation
- Contamination
- Collapses (impact)

In general, there is a wide range of possible disasters of technological origin. At present, urban centers and ports are highly susceptible to this type of event due to the high density of industry, buildings, and mass transportation of cargo and people.

EFFECTS OF DISASTERS

The effects that a disaster can cause vary in accordance with the characteristics of the exposed elements and the nature of the event itself. The impact may produce different types of disruptions. In general, the population, the environment, and physical structures as represented by housing, industry, commerce, and public services can be considered as elements at risk.

The effects can be classified as direct and indirect losses. Direct losses are related to physical damage, in the form of victims, damage to the infrastructure of public services, damage to buildings, the urban area, industry, commerce, and the deterioration of the environment, that is, the physical alteration of the habitat.

Indirect losses can usually be subdivided into social effects such as the interruption of transportation, public services, the mass media and the unfavorable image that a region can develop with respect to others; and into economic effects represented by disruptions of commerce and industry as a consequence of the drop in production, disincentives for investment, and retrofitting and reconstruction costs.

In a great many developing countries, such as the countries of Latin America and the Caribbean, there have been disasters in which thousands of people have died and losses in the hundreds of millions of dollars have occurred in twenty or thirty seconds. In many cases the figures are incalculable, and the direct and obviously indirect costs of these events can amount to a huge percentage of a country's gross domestic product. Due to the recurrence of different types of disasters, in several countries on the continent the losses due to natural disasters can amount to a significant average percentage of their annual gross national product. This situation obviously translates into impoverishment for the people and stagnation, because it entails unforeseen expenditures that affect the balance of payments and, in general, the economic development of these countries.

Preventive measures against the effects of disasters should be considered a fundamental element in the processes of comprehensive development at the regional and urban level, in order to reduce the existing level of risk. Since such events can have a serious impact on the development of these communities, preventive measures must be given priority over post-disaster recovery, and risk analyses must be incorporated into the social and economic considerations of each region or country.

CONCEPTUAL FRAMEWORK

The impact of disasters on human activities has been dealt with in recent years in many publications in various disciplines, which have conceptualized their components in a different, although in most cases similar manner. The United Nations Disaster Relief Organization

(UNDRO—currently United Nations Office for Humanitarian Affairs—UN/DHA), together with the United Nations Educational, Scientific, and Cultural Organization (UNESCO), sponsored a meeting of experts who proposed standardized definitions that have been widely accepted in the last few years. The report of this meeting, "Natural Disasters and Vulnerability Analysis," included the following definitions, among others:

Hazard (H), defined as the probability of occurrence of a potentially disastrous event during a certain period of time at a given site.

Vulnerability (V), defined as the degree of loss of an element or group of elements at risk as a result of the probable occurrence of a disastrous event, expressed on a scale from 0 (no damage) to 1 (total loss).

Specific Risk (R_s), defined as the degree of expected loss due to the occurrence of a specific event and as a function of the hazard and the vulnerability.

Elements at Risk (E), such as people, buildings, and public works, economic activities, public services, utilities, and infrastructure exposed in a given area.

Total Risk (R_t), defined as the toll of dead and wounded, property damage, and impact on economic activity from the occurrence of a disastrous event, that is, the result of the specific risk (R_s) and the elements at risk (E).

In other words, risk can be calculated in accordance with the following general formulation:

$$R_t = E \cdot R_s = E \cdot (H \cdot V)$$

In light of the elements at risk (E) implicit in vulnerability (V), which does not, however, modify the original concept, it could be posited that:

Once the hazard (H_i), understood as the probability that an event with an intensity greater than or equal to i will take place during an exposure period t, is known, and once vulnerability (V_e), understood as the intrinsic predisposition of an exposed element (e) to be affected by or to suffer a loss from the occurrence of an event with an intensity (i), is known, the risk (R_{ie}) can be understood as the probability that there will be a loss in the element (e) as a result of the occurrence of an event with an intensity greater than or equal to (i);

$$R_{ie} = (H_i, V_e)$$

in other words, the probability of exceeding a certain level of social and economic consequences during a given period of time (t).

More precisely, then, two concepts may be distinguished that on occasions have mistakenly been considered synonymous but that are definitively different both from the qualitative and quantitative points of view:

- ***The hazard***, or external risk factor of a subject or system, represented by a latent danger associated with a physical phenomenon of natural or technological origin that can occur at a specific site and at a given time, producing adverse effects on individuals, property, and/or the environment, mathematically expressed as the probability of exceeding a level of occurrence of an event with a certain intensity at a certain site and during a certain period of time.
- ***The risk***, or harm, destruction, or loss expected from the combination of the probability of occurrence of hazardous events and the vulnerability of the elements exposed to such hazards, mathematically expressed as the probability of exceeding a level of economic and social consequences at a certain site and during a certain period of time.

In general, "vulnerability" can be understood, then, as the intrinsic predisposition of a subject or element to suffer harm due to possible external actions. Therefore, evaluating it makes a fundamental contribution to knowledge of the risk through interactions of the susceptible element with the hazardous environment.

The fundamental difference between hazard and risk is that the hazard is related to the probability that a natural or induced event will take place, while the risk is related to the probability that there will be certain consequences, which are closely related not only to the degree of exposure of the elements but also to the vulnerability of these elements to the impact of the event.

HAZARD AND SEISMIC RISK

Earthquakes consist of sudden releases of energy due to stresses accumulated over the years at places in the earth's crust. The leading causes of stress in the crust are found in the forces that pull at the segments of which it is composed (the so-called tectonic plates), which are countered by opposing forces in the adjacent plates. Enough is not yet known about the nature of all these forces, but they may be due either to high temperatures inside the earth or to the force of gravity. The earthquakes so caused are usually of average to great depth.

The forces that develop in the tectonic plates produce, in turn, cracks within the plate itself, known as geological faults. Forces derived from tectonic activity can manifest themselves in the faults and tend to move a segment of the fault, generating a reverse reaction in the opposing segment of the fault, thus starting the process of energy accumulation. Earthquakes caused by active faults are in general of low or average depth and are consequently very hazardous.

The usual ways of measuring an earthquake are related to its energy, location, and manifestation on the surface in cities or sites of interest. The energy of an earthquake is measured as magnitude, which was developed by Charles Richter as a simple number that describes the energy released by means of the formula:

$$\log E = 11.8 + 1.5\, M$$

Seismographs are used to measure the magnitude and determine the location (epicenter) of the phenomenon. Magnitude as such is merely a measurement of the earthquake event itself at the site of energy release. At sites far from the event this energy is attenuated due to the damping properties of the rocks through which the seismic waves travel. For this reason it is more appropriate to measure the impact on sites of interest in terms of the ground movements themselves. This measurement, which is done with accelerometers, usually registers ground movement in the three spatial directions in terms of their acceleration, since this measurement includes information about speed and ground deformation.

Ground movement is, consequently, a function of the magnitude of the earthquake, the distance from the point where energy is released, and the properties of energy attenuation of the geological site. The studies of seismic hazard seek to establish, for each place of interest, an earthquake that has a low probability of being exceeded during a period deemed appropriate as the average life of the building or buildings to be constructed, in accordance with available information on the seismic sources that affect that site.

In addition to what has been mentioned previously, the following factors can influence the impact of an earthquake on cities:

- *The amplification of seismic waves through the ground.* This fact is currently receiving much attention from researchers, since the energy of earthquakes can be greatly amplified by the characteristics of the soils that supports buildings in cities. Earthquakes occurring at great distances from a city that are practically insignificant on hard or rocky surfaces are amplified destructively when the ground is soft, usually lacustrine.

- *Soil liquefaction.* In certain cases, especially in saturated sandy soils of uniform gradation, the phenomenon of soil liquefaction can occur, whereby the ground suddenly sinks because of an increase in the pressure of the water contained in the soil during a seismic vibration. This can be catastrophic.
- *Mass movements.* Mountainous terrain can suffer landslides or collapses as a consequence of the seismic thrust of the earth. On occasion, mass movements do not occur immediately after the earthquake but in several hours or days.
- *Ground settlement.* This can occur in not very compact surfaces or surfaces supported on layers of soil that have undergone liquefaction, etc.
- *Tsunamis or tidal waves.* Ocean waves generated by seismic activity on the ocean floor can cause flooding in coastal areas and affect other areas located thousands of kilometers from the location of the earthquake.
- *Indirect hazards.* The forces of the earthquake can cause dams to fail, which can aggravate the effects of the event downstream from reservoirs; or can cause contamination from damage to industrial plants, such as leaks of gases or hazardous substances, explosions, and fires.

Most of the damage caused by earthquakes is due to powerful ground movement. High magnitude earthquakes have been felt over areas on the order of 5 million square kilometers. For this reason, engineering decisions are normally made on the basis of evaluations of large movements, expressed in terms of the maximum acceleration that can be expected from the ground movement at each site.

Central America and South America, especially the Pacific coast, are areas of high seismicity and seismic hazard. Major earthquakes have occurred between Costa Rica and Panama (measuring 8.3 on the Richter scale in 1904), along the border between Colombia and Ecuador (8.9 on the Richter scale in 1906), in Peru (8.6 on the Richter scale in 1942), north of Santo Domingo, Dominican Republic (8.1 on the Richter scale in 1946), and in Chile (8.4 on the Richter scale in 1960). In general, all of the countries of Latin America show some degree of seismic hazard, given that their different provinces have seen earthquakes that while they are not recalled as events of great magnitude did frequently cause major catastrophes and damage. Approximately 100,000 inhabitants of this region have died as a result of earthquakes this century, and 50,000 as a result of volcanic eruptions, while the number of wounded greatly exceeds the death toll.

Hospitals and, in general, health facilities are exposed elements that can suffer serious damage as a consequence of strong earthquakes. In other words, the seismic risk to health facilities can be very high, which is why new buildings must be constructed in accordance with seismic-resistant standards adapted to the seismic hazard of each area. The seismic vulnerability of existing buildings must also be evaluated in order to identify their weaknesses and to design and carry out whatever physical modifications or retrofittings are necessary.

Table 1 shows a list of hospitals that have suffered very serious damage or structural collapse as a consequence of earthquakes.

HOSPITAL	COUNTRY	EARTHQUAKE
Kern Hospital	U.S.A.	Kern County, 1952
Hospital Traumatológico	Chile	Chile, 1960
Hospital Valdivia	Chile	Chile, 1960
Elmendorf Hospital	U.S.A.	Alaska, 1964
Santa Cruz Hospital	U.S.A.	San Fernando, 1971
Olive View Hospital	U.S.A.	San Fernando, 1971
Veterans Administration Hospital	U.S.A.	San Fernando, 1971
Seguro Social	Nicaragua	Managua, 1972
Hospital Escalante Padilla	Costa Rica	St. Isidro, 1983
Hospital Juárez	Mexico	Mexico, 1985
Centro Médico	Mexico	Mexico, 1985
Hospital Bloom	El Salvador	San Salvador, 1986
Hospital San Rafael	Costa Rica	Piedras Negras, 1990

TABLE 1. SELECTED HOSPITALS DAMAGED BY EARTHQUAKES IN THE REGION OF THE AMERICAS

CHAPTER 2

ANALYSIS AND DESIGN OF HOSPITAL BUILDINGS

INTRODUCTION

The structural analysis of hospital buildings does not differ in its theoretical aspects from an analysis of normal buildings, because they are constructed with identical materials and are subject to similar loads. The differences can be found, instead, in the specific design criteria and requirements for hospitals, on which there is no consensus in the international community, as this chapter will explain later.

Below we offer in summary form the basic principles of structural analysis adapted to buildings.

STATIC ANALYSIS

Structures subjected to static loads, such as those represented by the weight of the structure itself and live loads, tend to be modeled as hyperstatic, linearly elastic structures, for which the principle of superposition is valid. This makes it possible to relate the forces and the deformations through the simple formula:

$$f = k\,u$$

where f is the vector of external forces applied to the structure, u is the vector of deformations at the points of application of these forces (degrees of freedom), and k is the matrix of stiffness defined as:

$$k = [k_{i,j}]$$

in which the elements denote the force in i when there is a unitary displacement in j.

The usual analysis for gravity loads, as well as for loads arising from ground and water movements and differential settlements, uses the above equation, with the assumption of linearity in the stress-strain performance of the materials. This assumption is appropriate for materials such as steel and aluminum at moderate levels of stress, below the level of yielding, since the modulus of elasticity of the material is constant up to this level. For concrete and masonry the assumption is less appropriate, because the slope of the stress-strain curve varies with the level of stress, but the use of the equation is at any rate acceptable.

The elementary terms of stiffness utilized so far are usually modified to take into account the following facts:

- The influence of shear deformations, which is greater as the height to length ratio of the section increases.
- The high stiffness of every element in the area of the connection, which can be considered as infinite.

The matrix of stiffness of a prismatic bar obtained by taking into account these two modifications is the one that is usually entered into computer programs to analyze reticular structures. J. Meek (*1*) describes in detail structural analyses by matrix methods, as well as the systems of electronic calculation.

THREE-DIMENSIONAL ANALYSIS

The three-dimensional nature of buildings and of seismic loads has made it necessary to conduct analyses in three dimensions that reflect the behavior of structures more appropriately than plane analyses do.

The need for three-dimensional analysis is due basically to the phenomenon of torsion. Indeed, owing to the difference in the plane disposition of lateral stiffness, torsion arises under the presence of seismic or wind inertial forces, which, in simplified terms, because they are inertial forces have their resultant applied in the center of mass of every level. Owing to the difference of position between the center of mass and the point of concentration of the resultant of the reaction forces of the different resistant elements, an instantaneous torsion of the diaphragm is generated, and it increases as the distance between the two points increases. The point of application of the aforementioned resultant is known as the center of rotation, and its position is, in general, variable, depending on the load, the resistance actually offered by the elements in each case, etc. For the conversion of three-dimensional analysis into a series of plane analyses, it can be presumed that this center is located approximately in the center of stiffness of the different elements that cross the diaphragm (2). If a three-dimensional analysis like the one explained later is conducted, this assumption is not necessary.

Static three-dimensional analysis

The following mathematical elaboration assumes that the floor slab joins the resistant vertical structures (frames and walls) without absorbing deformations, that is, rigidly. This presumes that the seismic forces are absorbed entirely by the vertical structures and that the slab is infinitely rigid in its plane. This assumption, which is discussed in greater detail in chapter 3, in in accordance with what is observed in most practical cases. Figure 1 illustrates this rigid behavior in a structure without torsion, as well as a case of flexible behavior by the slab in its own plane, which is considered undesirable. It can be clearly seen that this case would require a three-dimensional analysis that considers the flexibility of the slab in its plane. Other cases of this type of behavior are described in chapter 3.

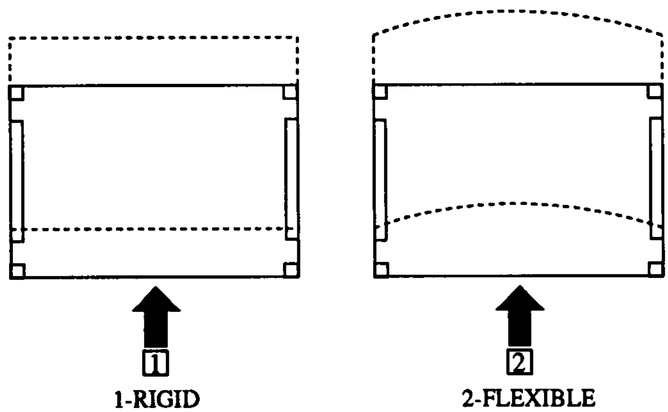

FIGURE 1. RIGID AND FLEXIBLE BEHAVIOR OF THE DIAPHRAGM

Let us assume that a plane system ***i***, subjected to lateral loads, is analyzed in its two co-ordinates, utilizing the condensed matrix, in the equation:

$$f_i = R_i \, u_i$$

The vector of forces can be expressed in a spatial system of co-ordinates, as:

$$F_i = A_i \, f_i$$

where matrix A_i is necessary for conveying the vector of forces in the local system to the center of co-ordinates of the spatial system. Under the law of reverse grade, this matrix is the same that is required in order to express the relation between the displacements in the plane system and the spatial system:

$$u_i = A_i^T \, U_i$$

where U_i is the vector of displacements of the plane structure seen from the spatial system. Replacing:

$$f_i = R_i \, A_i^T \, U_i$$

and premultiplying by A_i,

$$A_i \, f_i = F_i = (A_i \, R_i \, A_i^T) \, U_i$$

or, the equivalent:

$$F_i = S_i \, U_i$$

with:

$$S_i = A_i \, R_i \, A_i^T$$

where S_i is the matrix of stiffness of the plane structure expressed in terms of the system of spatial co-ordinates. Similarly, by conveying all the matrices of stiffness of the plane systems to the common center, one obtains:

$$S = \sum_{\forall i} S_i$$

The seismic forces calculated by means of dynamic analysis or the simplified method, which are described below, are considered to be applied at the gravity center of each floor, owing to their nature as inertial forces. Vector $\boldsymbol{F}$ is the vector of these forces:

$$F = [\ f_j\ ]$$

Displacements in the overall system produced by these forces will be:

$$U = S^{-1}F$$

The displacements in each of the plane systems will be obtained as:

$$u_i = A_i^T\ U$$

and the rotations of the connections would be calculated in accordance with the equation:

$$\theta i = -K_{\theta\theta}^{-1}\ K_{u\theta}\ u_i$$

With the two results the internal forces in the elements of the plane system are easily calculated.

DYNAMIC ANALYSIS

Dynamic analysis must be used for seismic loads. For the simple case of undamped linear vibrations without an external acting force, that is, free vibrations, in a system with one degree of freedom, there is:

$$m\ddot{u} + ku = 0$$

which has as a solution:

$$u(t) = e^{-\xi\omega t}\ (A\ \sin\omega_d t + B\ \cos\omega_d t)$$

with:

$$\xi = \frac{c}{2m\omega} \equiv \textit{damping factor}$$

$$\omega = \sqrt{\frac{m}{k}} \equiv \textit{natural frequency}$$

$$\omega_d = \omega\,\sqrt{1-\xi^2} \equiv \textit{damped frequency.}$$

The forced vibration, yielded by:

$$m\ddot{u} + c\dot{u} + ku = p$$

has as a solution the Duhamel integral:

$$u(t) = \frac{1}{m\omega_d}\int_0^t p(\tau)\ e^{-\xi\omega(t-\tau)}\ \sin\omega_d(t-\tau)\,d\tau$$

The histories of velocity and acceleration of the system are obtained by numerical derivation.

In the case of ground movements, the dynamic force has as its value:

$$p(t) = -m\ddot{u}_s$$

where:

$$\ddot{u}_s \equiv \textit{ground acceleration.}$$

Accordingly,

$$u = -\frac{1}{\omega_d}\int_0^t \ddot{u}_s\ e^{-\xi\omega(t-\tau)}\ \sin\omega_d(t-\tau)\ d\tau$$

A detailed examination of the dynamics of structures can be found in Clough and Penzien (*3*).

Response spectra

The repeated application of Duhamel's integral for a single history of ground acceleration and several damping factors and frequencies can yield multiple response histories, whose maximum values tend to be plotted against their respective frequencies or periods in order to get an idea of the earthquake's action on several types of structures. These spectra are defined as:

$$S_d(T,\xi) = \max\ u(t,T,\xi)$$

$$S_v(T,\xi) \equiv \max\ \dot{u}(t,T,\xi) \approx \omega\ S_d(T,\xi)$$

$$S_a(T,\xi) \equiv \max \ddot{u}(t,T,\xi) \approx \omega^2 S_d(T,\xi)$$

The higher value of the accelerations in a given area of periods is due to the fact that the natural frequencies in the area approach the dominant frequencies of the seismic waves in the foundation, which is a necessary condition for a sort of resonance of the system through excitation. The seismic waves in the foundation of the structure, in turn, have been filtered through the soil overlying the basal rock and have acquired accordingly the dominant frequencies of the soil. For this reason one should consider with care the ratio of the natural dominant frequency of the structure to the dominant frequencies of the soil. The same structure, located on two different types of soil, can respond with greater accelerations in one than in the other, depending on the ratio of its natural frequency to that of the soil (*4*).

Figure 2 shows several spectra of earthquake records obtained in different types of soils, standardized according to the maximum acceleration of the ground. It can be clearly seen that the effect of soil stiffness is twofold. On the one hand, it determines the area of the spectrum in which the greatest response values occur, so that the greater the flexibility of the soil, the greater the dominant period in the spectrum. On the other hand, the greater flexibility of the soil causes a greater amplification of acceleration in the structure.

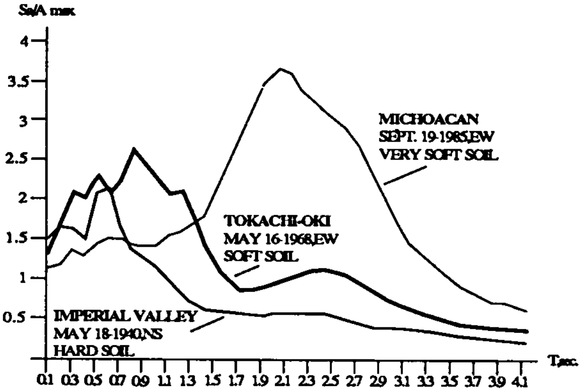

FIGURE 2. SPECTRA OF ACCELERATION: COMPARISON OF SITE EFFECTS

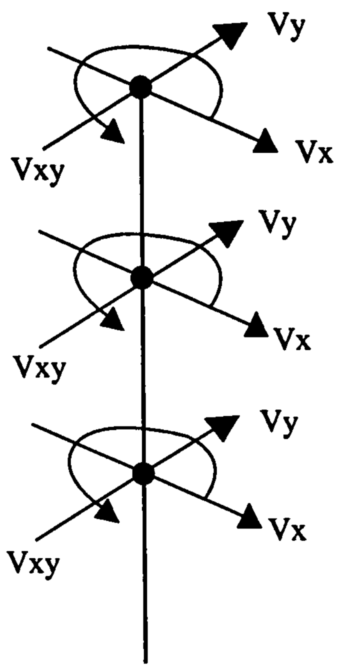

FIGURE 3: DYNAMIC THREE-DIMENSIONAL MODEL

Multidegree of freedom systems

The equation for nondamped free vibration in multidegree of freedom systems looks like:

$$M\ddot{U} + S\,U = 0$$

where:

$$M = [m_{ij}] \quad , \quad S = [s_{ij}]$$

are the matrices of mass and stiffness, respectively. The terms of the matrices are the forces in the degree of freedom i due to a unitary acceleration or to a unitary displacement, respectively, in the degree of freedom j. The model normally adopted is shown in figure 3. The masses have as degrees of freedom two lateral orthogonal translations and a rotation around the central axis. This appropriately simulates the displacements of the structure in the horizontal plane and the torsion of the structure caused by eccentricities of the mass with respect to stiffness. The matrix of stiffness, consequently, is constructed in accordance with these degrees of freedom, as illustrated later on. However, it may be sufficient to use a plane model for each horizontal axis, and to analyze the torsion effects in a static three-dimensional analysis with the forces resulting from this dynamic analysis. In this case, the matrix of stiffness is the total of the condensed matrices of stiffness obtained for each resistant frame in the direction of the analysis:

$$S = \sum_{k=1}^{k=N} R_k$$

If the following solution is assumed:

$$U = \phi \sin(\omega t + \gamma)$$

we obtain:

$$[S - \omega^2 M]\,\phi = 0$$

which has a solution only if:

$$\| S - \omega^2 M \| = 0$$

The solution for this determinant gives the different vibration frequencies, to each of which corresponds a vibration mode. Physically, the significance of the vibration mode is that, if the structure is carried to the point of deformation indicated for each mode and is released, it will vibrate freely at the corresponding frequency, taking at each extreme of the vibration the form of the respective mode.

Dynamic forces act through the dynamic characteristics of the structure, that is to say, its frequencies and modes of vibration, each of which represents a system of one degree of freedom. For this reason, the total response for forced vibrations, according to the principle of superposition of linear systems, will be the total of the individual responses. The total value of the displacement response is given by:

$$U = \phi_1 V_1 + \phi_2 V_2 + \ldots + \phi_n V_n$$

where the first factor of the terms in the series is the mode of vibration and the second represents the response of said mode to excitation and is known by the name of generalized coordinate of the respective mode. Thus:

$$V_i = \frac{1}{M_i \omega_{d_i}} \int_0^t P_i(\tau)\ e^{-\xi_i \omega_i (t-\tau)}\ \sin\omega_d(t-\tau)\ d\tau$$

where:

$$M_i = \phi_i^T\, M\, \phi_i$$

and:

$$P_i = -\ \phi_i^T\, M\ [r]\ \ddot{a}_s$$

where **[*r*]** is a vector of influence that represents the static displacements in the degrees of freedom of the structure that result from a unitary static movement in the foundation. For the plane models vector **[*r*]** is a vector of several planes.

In accordance with the equations for the dynamics of structures it can be demonstrated that the seismic force in mode ***i*** and at level ***j*** is:

$$f_{i,j} = \frac{\sum_{\forall j} m_j \phi_{i,j}}{\sum_{\forall j} m_j \phi_{i,j}^2}\ m_j\ \phi_{i,j}\ S_a(T_i, \xi_i)$$

The maximum forces due to each mode do not necessarily coincide in time, due to the difference of response of each mode, which in turn is due to the difference of frequencies. Thus, their combination to obtain the design force tends to be probabilistic. The usual way to carry out this combination is by means of the square root of the sum of the squares:

$$f_j = \sqrt{\sum_{\forall i} f_{i,j}^2}$$

In recent years, however, the so-called complete quadratic combination has acquired greater application, in which:

$$f_j = \sqrt{\sum_{i}^{n} \sum_{k}^{n} f_{ij}\, p_{ik}\, f_{kj}}$$

with:

$$p_{ik} = \frac{8\sqrt{\xi_i \xi_k \omega_i \omega_k}\,(\xi_i \omega_i + \xi_k \omega_k)\,\omega_i \omega_k}{(\omega_i^2 - \omega_k^2)^2 + 4\xi_i \xi_k \omega_i \omega_k (\omega_i^2 + \omega_k^2) + 4\,(\xi_i^2 + \xi_k^2)\,\omega_i^2 \omega_k^2}$$

Simplified method

From the previous formulas can be derived a simplified method of calculating seismic forces. The force due to mode ***i*** on floor ***j*** can also be expressed as:

$$f_{i,j} = \frac{\left(\sum_{\forall j} m_j \phi_{i,j}\right)^2}{\sum_{\forall j} m_j \phi_{i,j}^2} \; \frac{m_j\, \phi_{i,j}}{\sum_{\forall j} m_j\, \phi_{i,j}} \; S_a(T_i, \xi_i)$$

The first term is known by the name of effective modal mass of mode ***i***. The usual tendency in buildings is for the effective mass of the first mode of translation in each direction to constitute most of the total mass. If one takes the effective mass of the first mode as the total mass of the building, and if it is assumed that the first mode has a linear form, expressed as the ratio of the height of the floor to the total height of the building, so that:

$$\phi_{1,j} \doteq \frac{h_j}{H},$$

one obtains:

$$f_j = \frac{w_j h_j}{\sum_{\forall j} w_j h_j} \; \frac{W}{g} \; S_a(T_1, \xi_1)$$

which is the basic formula for calculating forces by the pseudo-dynamic methods set forth in several construction codes. The method manages to completely avoid a dynamic analysis by assuming a fundamental period in accordance with empirical formulas calculated by experimentation on buildings with environmental vibrations. The supporters of the method defend it on the grounds of its simplicity. However, when the complexity of the structure increases, dynamic analysis becomes more necessary, since the fundamental mode tends to depart from the form taken by the simplified method, and the superior modes take on greater importance.

SEISMIC-RESISTANT DESIGN

The seismic-resistant design of structures is much more complex than the design for static gravity loads, due to the multiple factors that must be taken into account. Among them are the following:

a) The random nature of excitation, in terms of the moment of its occurrence, location, energy released, amplification by the soil, variation over time, etc.
b) The uncertainty as to the response of the structure, due to the heterogeneous quality of the materials, the interaction with non-structural elements, the variation of the live loads, the variations in construction, etc.
c) The failure and energy dissipation mechanisms that entail the least risk for human life and property.
d) The extra cost, on top of resistance to gravity loads, entailed in avoiding total or partial structural and non-structural damage.
e) The cost of repairs if a given level of damage is allowed.
f) The social cost entailed in the failure of buildings, especially if they are essential to cope with a disaster, as in the case of hospitals.

Accordingly, a seismic-resistant design should try to take all of these considerations into account (*5*). Normally, design codes address some of these problems by means of simple quantitative formulas on overall or local safety. Strict compliance with these standards in the customary design of structures means that the underlying substance of such simplifications is frequently ignored or forgotten, which gives rise to routine, thoughtless design work. However, in the design of any building, especially ones that must remain in the best possible condition after an earthquake, the implications of each important decision must be borne in mind in accordance with the principles and advances of seismic engineering and in light of the fact that the building has a vital role in maintaining society's well being.

This chapter reviews the implications of the aforementioned considerations for the seismic design of hospitals.

Design spectrum

The design spectrum recommended in seismic codes addresses:

a) *The probability of exceeding the design earthquake in a period of time considered as the average useful life of the buildings*. Normally, this probability is considered to be 10% over a useful life of 50 years. In the case of hospitals, however, useful life far exceeds this number. The frequency of hospital construction is decidedly less than that of the construction of housing and other types of structures. This is especially critical in the developing countries, in which few large hospitals are built since constructing them is a major burden, for various reasons, and usually runs a deficit. For these reasons such centers are meant to last a very long time in some countries, and accordingly, careful thought should be given to the selection of these variables.

b) *The dominant frequencies and maximum responses*. Normally, the spectra of earthquakes exhibit narrow frequency ranges in which the maximum responses are found. However, to dispel the uncertainties associated with the distance at which the event occurs and its content of frequencies, the design spectra display a broad range of maximum responses as well as amplification factors of the responses in soft ground with respect to responses on firm ground, in accordance with the behavior observed in several sites around the world. However, in the case of special buildings, it may be desirable to prepare a design spectrum in accordance with the geological and geotechnical characteristics of the construction site.

Non-linear behavior

The traditional design criterion for buildings subject to strong earthquakes has been to allow the materials some degree of non-linear response for the purpose of absorbing energy through permanent deformations. Figure 4 illustrates this criterion for an elasto-plastic system. The line OA represents the maximum stress-maximum strain diagram of a perfectly elastic system during a given earthquake, while the line OCD represents an elasto-plastic system. There are several hypotheses as to the simplification that should be assumed in order to evaluate simply the performance of the elasto-plastic system.

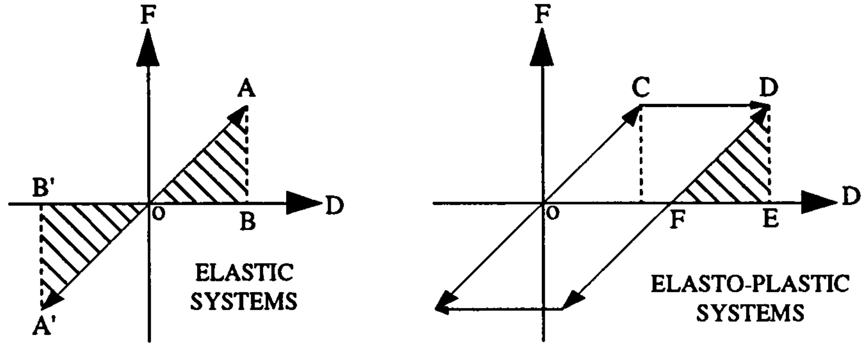

FIGURE 4. ABSORPTION AND DISSIPATION OF ENERGY

Hypothesis of equal maximum deformation. If we assume that an elasto-plastic system, OAB (figure 4) has the same stiffness in its elastic part as the elastic system, this means that the initial period of the elasto-plastic system is equal to the period of the elastic system. If we assume in addition that the elastic and elasto-plastic systems have during the same earthquake one equal maximum deformation, OE=OB, then the following ratio is valid:

$$\frac{AB}{DE} = \frac{OE}{OF}$$

That is,

$$\frac{f}{f_y} = \frac{\delta_u}{\delta_y}$$

If we make:

$$\mu = \frac{\delta_u}{\delta_y}$$

and we define it as a factor of ductility, the following ratio expresses the design forces of the elasto-plastic system:

$$f_y = D\, f$$

with:

$$D = \frac{1}{\mu}$$

Factor ***D*** is usually called the stress reduction factor. Thus, the structure should be designed for less stress than is produced by the response of the elastic system, calculated in accordance with what was stated in the first part of this chapter. If the forces thus obtained are used to conduct an elastic analysis, some deformations would be obtained that, in turn, should be multiplied by the ductility factor in order to estimate the maximum deformations of the structure, this being of great importance for the study of the behavior of non-structural elements and the stability of the different floors. The structural elements should then guarantee that these inelastic deformations are achieved. For this purpose these elements should have sufficient ductility, by means of the mechanisms that will be detailed further on.

Hypothesis of equal energy of monotonic deformation. In this case we will assume that the potential energy stored in the two systems, which are of equal initial stiffness and are loaded monotonically, is equivalent. The area OAB (figure 4) represents the elastic energy stored in the elastic system, which becomes kinetic energy that impels the structure for additional displacements in the opposite direction, while the area FED represents the part of the energy stored in the elasto-plastic system that is capable of generating additional displacement. It can be seen that since the resistance of the system is lower, the elasto-plastic system should suffer slower accelerations than the elastic system. On the other hand, the area FCDE represents the energy absorbed by the elasto-plastic system through permanent deformations. Equalizing the two total energies, we have:

$$\frac{OA \ x \ OB}{2} = \frac{FC \ x \ OF}{2} + FE \ x \ FC$$

That is,

$$\frac{f\delta}{2} = \frac{f_y \delta_y}{2} + f_y(\delta_u - \delta_y)$$

as:

$$OD = OC \ x \ \frac{OG}{OF}$$

we arrive finally at:

$$D = \frac{f_y}{f} = \sqrt{\frac{1}{2\mu-1}}$$

with:

$$\mu = \frac{\delta_u}{\delta_y}$$

Accordingly, the maximum deformation of the elasto-plastic system is yielded by:

$$\delta_u = \delta_y \; \mu$$

Thus, different equations for the reduction of forces must be applied in the design of the elasto-plastic system, as well as for its maximum deformation. Rosenblueth (*6*) recommends using the reduction of forces according to the first hypothesis for long periods and the second for short periods. For estimating inelastic displacements the recommendation is to utilize the most conservative hypothesis in every case.

It should be noted that these design hypotheses are established in many construction codes. However, they have the defect of considering a reduction of forces due to inelastic behavior only in connection with the maximum deformation at any instant of the earthquake or to the maximum energy dissipated in a cycle, without taking into account its duration. As a result, factors as important as those associated with the progressive fatigue of the materials are disregarded, such as the degradation of stiffness, the reduction of resistance, the steady increase in deformations, and thus, progressive collapse. For this reason, in recent years major emphasis has been placed on the methods that in one way or another take into account the total duration of the earthquake in the design, usually through total dissipated energy or the number of load cycles.

Drifts and stability

In principle, large lateral displacements endanger the safety of the structure in its entirety, due to the damage that they can cause to non-structural elements in general. However, when they are even larger, they carry with them the risk of the partial or total collapse of the building (figure 5).

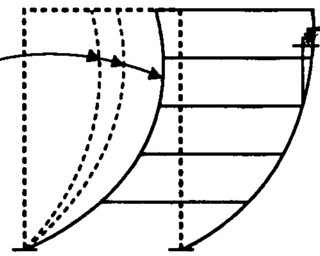

FIGURE 5. DRIFT AND STABILITY

The damage to non-structural elements attached to the structure is particularly serious in the case of hospitals, for which reason this subject will be treated specifically in chapter 3. For the time being, it is necessary to bear in mind that this damage is associated with the value of the relative inelastic displacement of one level with respect to the immediately contiguous one, or drift. It has been established that drift values higher than 1 or 1.5 per thousand of the clear height between the two levels are not desirable. However, this limit depends heavily on the fragility and resistance of the materials that make up the non-structural elements.

With respect to instability and second order effects, it can be said succinctly that they lead to an increase in floor drifts when the value of these effects is low, to an obvious increase in stress on the structural elements at the intermediate levels, and to the collapse of the floor (and perhaps, consequently, of the entire building) at its upper levels. A simple way to evaluate the degree of stability of each floor of a structure is by means of the index of overall instability effects:

$$\delta_g = \frac{1}{1-Q}$$

where:

$$Q_i = \frac{\sum_{j=i}^{n} P_{uj}\,\Delta_i}{V_i\,h_i}$$

In this equation,

P_{uj}=increased weight of story j, including live loads

$$\Delta_i = \mathit{Drift\ of\ story\ i}$$

$$V_i = \mathit{seismic\ shear\ on\ story\ i}$$

$$h_i = \mathit{elevation\ of\ story\ i}$$

The value of Q thus relates the increase in the moment of overturn of the floor, based on a brief second order analysis, with the direct or first order moment of overturn. It can be considered that for values of Q smaller than 0.3 the floor is stable, while for values greater than 0.5 it is definitely unstable. Figure 6 shows qualitatively the behavior of the amplification factor due to overall stability with respect to index Q.

Based on the above, calculating appropriate values of inelastic displacement is of great importance for a suitable analysis of the problems of drift and stability. As has been seen previously, and in view of the fact that the usual values of the reduction coefficient fluctuate between 0.17 and 0.5, the calculation of maximum displacement using the hypothesis of equal total energy with a constant yielding displacement seems more appropriate for design purposes, since it yields more conservative values. Being conservative in this regard is more desirable in the case of hospitals than in the case of other structures, due to the implications that the damage to non-structural and structural elements has for occupants and the community in general.

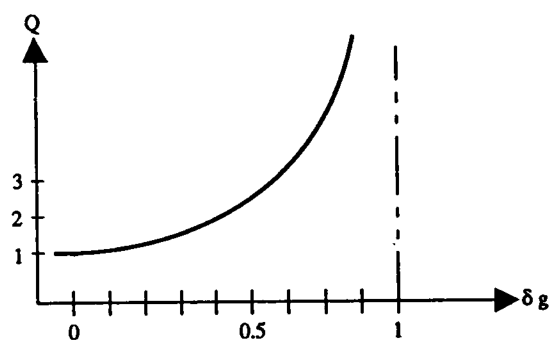

FIGURE 6. STRESS AMPLIFICATION FACTOR DUE TO OVERALL INSTABILITY

Optimizing safety and economy

Every engineering design should seek an appropriate balance between the desired safety conditions and the economic implications of providing such conditions. In the case of seismic designs, the criterion for establishing safety conditions varies in accordance with the type of structure to be built. Thus, in the case of office and residential buildings, the protection of human life can be established as a sufficient safety standard, accepting some structural and non-structural damage, while in the case of a nuclear power plant, the protection of the equipment is of utmost importance, all the more so if the consequences of their failure are taken into account (7).

Hospitals in particular must be designed in accordance with safety conditions that are pertinent not only to the safety of the people who may occupy them at the time of an earthquake, but also to the need to provide medical care to the victims of strong earthquakes over a wide region. The greater a health center's area of coverage in the event of disasters, the greater should be its level of safety. In general, the determination of a hospital's capacity should take into account the sudden potential increase in the demand for its services in such an event. This is illustrated in figure 7. The dotted line indicates the capacity of the hospital, and the thick line demand, both in normal times. A design for normal cases is appropriate if, as shown in the figure, the installed capacity exceeds normal demand. However, this may not be the case in many health centers in developing countries. The problem is worse in the case of disasters that affect the infrastructure of the hospital itself. At the moment of a seismic disaster demand increases abruptly, while service capacity can decrease as a result of serious damage to the structure, the cutoff of electricity, gas, and water, damage to equipment and facilities, the collapse of walls, ceilings, etc., due to decisions or omissions in the

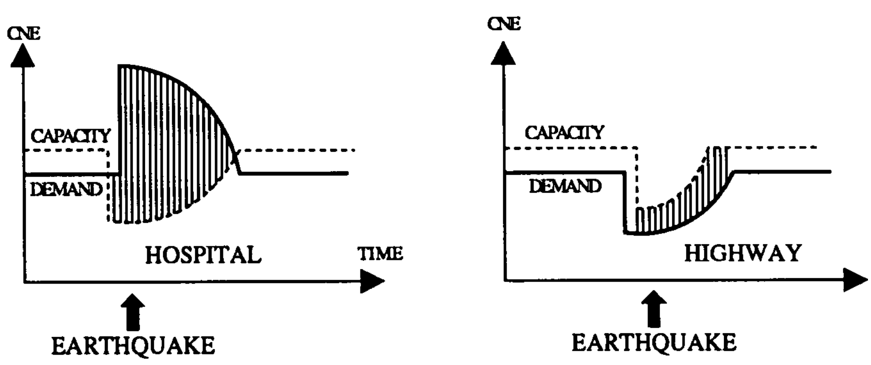

FIGURE 7. THE DEMAND-CAPACITY RATIO

design of the hospital. The gap between increased demand and reduced capacity can widen if the capacity of the hospital (considered in every aspect, such as medical personnel, beds, drugs, equipment, etc.) is inadequate even in normal times, which is also common in developing countries.

To provide a contrast, Figure 7 also shows the curves for a public service such as the road system, whose capacity may decline after a strong earthquake, but which, due to the overall economic impact on the region, also registers a reduction in demand. For this reason, in such cases the gap between demand and capacity after a strong earthquake is not as serious as in the case of hospitals.

Finally, there is a descending demand curve after the earthquake due to the care given to victims, but only as long as several strong earthquakes do not occur in a short period, while the capacity curve rises, as a function of the recovery of the hospital. The two curves are interrelated, since a reduction in demand over time depends on the recovery of capacity. Again, the economic situation of the region that is affected by damage to the hospital center has an impact on the recovery of capacity, which consequently may be more or less rapid.

All of the above means that considerations regarding the level of design of hospitals should be more or less strict depending on their social responsibility and level of development in each case. It is not appropriate to extrapolate criteria for the seismic design of hospitals from one country to another, not even from one region of a country to another, without considering the impact that a given level of damage has on each of them. Hence, so-called acceptable damage or risk should be established in each case as a planning decision by the society involved.

All of this affects the engineering design. For decision-making purposes, the total cost of a hospital can be estimated as:

$$TC\ (D) = ESC\ (D) + CNE\ (E) + CR\ (D) + HC\ (D)$$

where the terms indicate the extra structural costs (ESC) over and above those required merely for stability under gravity loads, due to the enhancement of the design level over a purely elastic seismic design that does not envisage damage, theoretically; the cost of the non-structural elements (CNE) in general; the cost of repair (CR), and, finally, the human cost (HC), understood as a conversion to economic values of the toll of dead and wounded. All these costs, except the second, can be considered to be dependent on a single variable, ***D***, that is, the coefficient of design forces reduction with respect to an elastic design.

The extra structural cost should take into account the increase in the design stresses of the elements, their stiffness, and their control during construction. Obviously, this cost must increase with factor ***D***. The cost of the non-structural elements is relatively impervious to factor ***D*** and increases only slightly with an increase in the levels of safety of equipment, facilities, and walls, which we can call ***E***. At any rate, it is a cost that can reach very high levels, on the order of 60 to 80% of the initial cost of the hospital. The cost of repairs should include not only the costs of physical repair of the building, but also the costs of replacing the affected equipment and facilities, as well as the impact on the time of recovery from such damage. In contrast to the above, this cost can be expected to diminish given the level of safety provided to the structure. Finally, the human cost should include not only the potential victims who are in the hospital but also, and more importantly, the wounded who can be expected to be sent to the hospital in the event of a seismic disaster, in accordance with the earthquake preparedness figures.

The last three costs are the ones that clearly differentiate the seismic design of a hospital from the design of an ordinary residential, office, or commercial building. Figures 8, 9, and 10 illustrate these differences.

While the extra structural cost curves may be the same or almost the same for the two cases, the curves for the non-structural elements are strikingly different, due mainly to the value of the equipment and hospital facilities, and show an increase with the level of safety provided to them, ***E***. As for the cost of repairs, the slope of the curve should in principle be steeper and show higher values for hospitals than for ordinary buildings, due more to the implications of damage to equipment and installations than to the repair of structural damage. Lastly, the human cost is radically greater in hospitals than in ordinary buildings for the reasons noted.

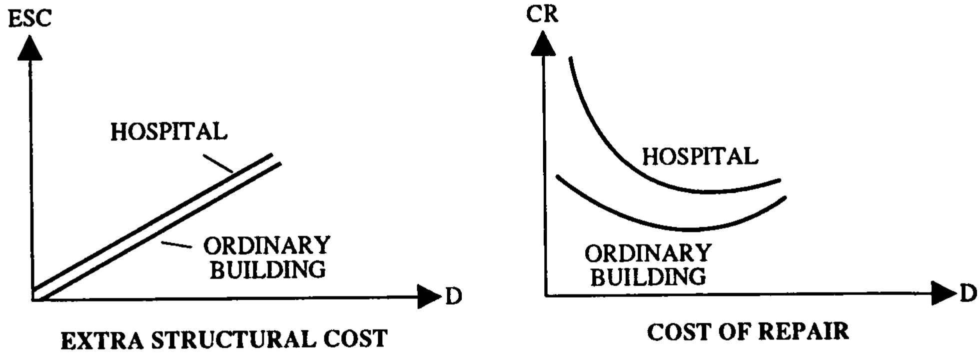

FIGURE 8. COST PATTERN I

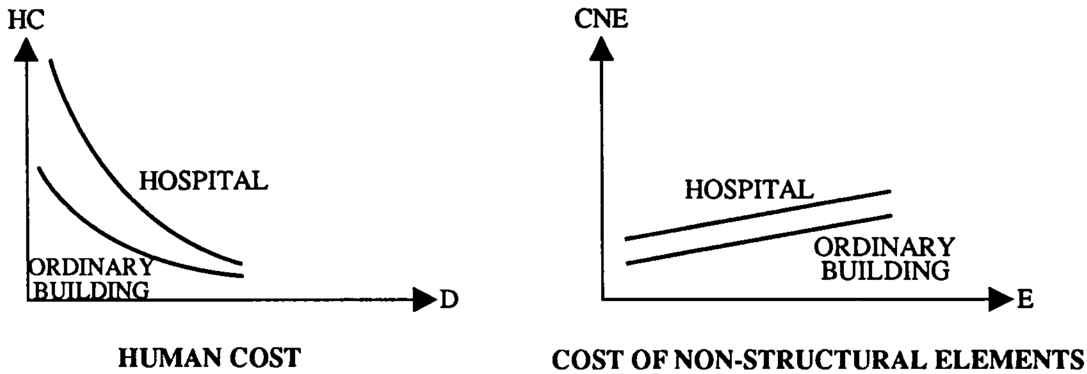

FIGURE 9. COST PATTERN II

In calculating the total cost for both cases and comparing it with the extra structural cost, which directly governs design, we can see in figure 10 that the point of lowest total cost is obtained for ordinary buildings with a design strategy that is more tolerant of damage, while for hospitals this point is reached with a less liberal strategy.

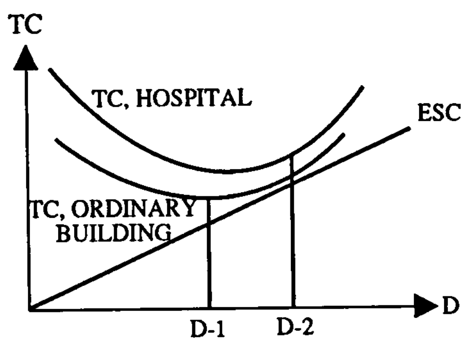

FIGURE 10. TOTAL COST

In principle, cost studies such as the one described above should be conducted as part of the process of planning a major hospital in seismic areas, in order to make appropriate decisions as to the level of structural safety that should be provided. As for the level of non-structural safety, the guidelines described in chapter 3 are sufficient.

Energy distribution

The estimates of non-linear behavior described previously have the disadvantage that they encourage an extrapolation of the results from a system of one degree of freedom to systems of multiple degrees, such as buildings. This is the design procedure recommended in many construction codes, which incorporate the use of an overall ductility factor for the entire structure, when actually the damage to or collapse of many structures shows that certain of their parts enter the inelastic range, beyond their capacity for ductility, while others remain in the elastic range. This makes the indiscriminate use of the overall ductility factor for various types of structures debatable, if no consideration is given to their conditions of mass distribution, resistance, or stiffness.

The method of total energy developed by Akiyama (*8*) is of great use in determining the distribution of energy absorption in the structure, in general, for the purpose of detecting its weak areas. In principle, an ideal design would be one in which the energy is absorbed in the form of inelastic deformations in similar proportion and form for all floors of the structure, so that each receives the least possible. In this case, the ductility factor reached in the earthquake would be similar on all floors. However, some circumstances can alter this uniformity and necessarily concentrate the energy on certain floors, which then achieve ductility factors that are much higher than expected, while the remaining floors may remain elastic, and thus the energy dissipation capacity of the latter is not taken advantage of. Among these circumstances are:

- Irregular distribution of mass, normally with the presence of some floors that are much heavier than others.
- Irregular distribution of stiffness, similar to what has been said about mass.
- Distribution of resistance in a way that departs greatly from a criterion deemed optimum for the development of a ductility demand that is similar on all floors.

Also, the floors that absorb large proportions of energy may contain elements that absorb more energy than others, thus causing the damage to be directed toward them.

The energy method utilizes the energy spectrum imparted to the structure by the earthquake instead of the usual spectra:

$$E = \int_0^s m\, \ddot{y}_o\, \dot{y}\, dt$$

where:

$$m = system\ mass$$

$$\ddot{y}_o = ground\ acceleration$$

The energy spectrum is defined more appropriately in terms of an equivalent velocity:

$$\dot{y} = system\ velocity.$$

$$V_e = \sqrt{\frac{2E}{m}}$$

As can be seen, the energy induced by the earthquake depends not only on the ground acceleration but also on the speed of the system, which can be elastic-damped or non-linear. A major conclusion of the study of energy spectra is that the inelastic spectra of various types of non-linearity (elasto-plastic systems, sliding systems, and combinations) are near in value to the energy spectrum of an elastic system with a damping of 10%. This greatly simplifies the calculations.

The energy induced in the structure breaks down into the following parts:

$$E=W_p+W_e+W_h$$

where W_p is the energy absorbed by the plastic yield of the structure, W_e is the elastic response energy of the system, and W_h is the energy consumed by the damping.

According to Akiyama (*8*), the distribution of energy absorbed by the structure through inelastic deformations on **i**-nth floor is a proportion of the total plastic yield energy mentioned above, with a value:

$$\frac{W_{p_i}}{W_p} = \frac{s_i p_i^{-12}}{\sum s_i p_i^{-12}}$$

where factor p describes the deviation of the seismic coefficient of the **i**-nth floor from a value deemed optimum for the simultaneous development of equal ductility on all floors. The variable s comprises the data on mass, stiffness, and capacity of inelastic deformation of every floor of the structure.

The form of energy distribution sheds a great deal of light on the weakest floors of the structure, where the energy may be concentrated and accordingly, where collapse may be more likely. In order to establish the possibility of collapse, it is necessary to calculate the ductility demand of the floor under analysis, which depends on the energy absorbed by it.

Figure 11 shows the results of five examples of 6-floor buildings evaluated by this method for ground accelerations on the order of 0.25 g. Case 0 is that of a frame in which the coefficient of seismic design of each floor (defined as the relationship between the shear design and the weight accumulated up to that floor) is the optimum one according to the method (in accordance with regression analysis). It bears noting that the values of this optimum coefficient differ greatly from the ones obtained from the conventional design formulas of the quasi-static and dynamic methods on upper floors. The stiffness of the frame has diminished gradually upward, in accordance with the lesser resistance required, which is also considered advisable. It can be seen that the percentages of energy distribution are similar on all floors.

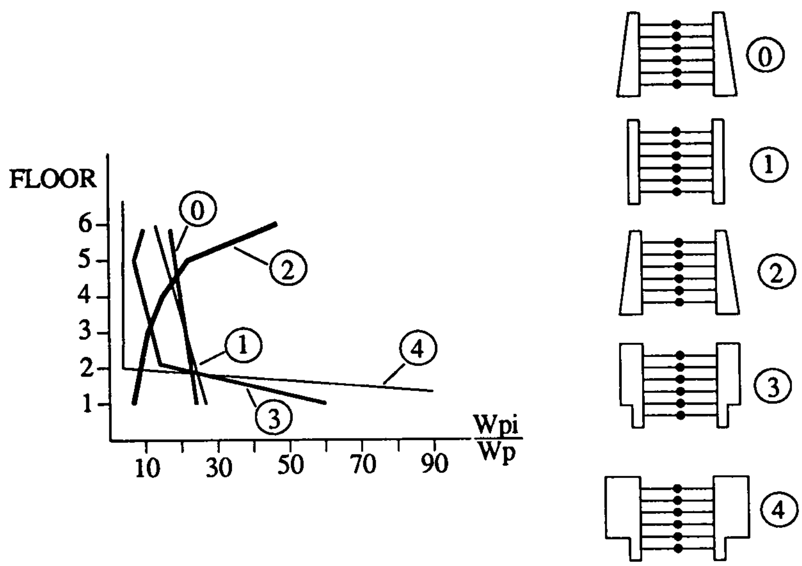

FIGURE 11. CASES OF ENERGY CONCENTRATION

Case 1 concerns a frame in which stiffness and mass have remained constant throughout the entire height. In case 2, a mass five times greater has been placed on the top floor than on the remaining floors. Case 3 displays a stiffness on the upper floors three times greater than on the first floor, and in case 4 that difference has been increased to thirty times.

The results obtained indicate clearly the departures from optimum behavior. In case 0, the absorbed energy increases mildly toward the first floor, which is desirable due to the increase in stiffness in the same direction. In case 1, this increase becomes sharper. In case 2, there is a major increase in the energy absorbed on the top floor due to the high concentration of mass there. In cases 3 and 4 there are sharp increases in the absorption of energy on the first floor, caused by the reduction in stiffness there. It can thus be seen that the results of the method agree with the observations made during different earthquakes as to the behavior of irregularly shaped buildings.

Provision of ductility

Since the methods of simplified non-linear design seen previously require that the structure be able to permit large deformations without collapse, the elements of the structure must be designed to meet this ductility demand appropriately.

Below we will examine the essential mechanisms for obtaining high ductility capacities in ordinary structural systems.

Steel structures. Unlike concrete, steel is by nature a ductile material. However, the major ductility demands imposed by earthquakes make it necessary to ensure certain special measures in the structural elements. The most important of them is the appropriate design of the beam-column connections. The beam-column connections are the most vulnerable points in steel structures subjected to earthquakes. The most common failure is due to local buckling in the flanges and core and tends to occur in the column at its connection with the beam, due to the forces transmitted by the latter. The design of the connection should be such that inelastic rotation takes place preferably in the beams, not at the connection. For this purpose the use of horizontal and/or inclined stiffeners may be required in the core of the column to control the transfer of stress from one element to the other.

Reinforced concrete structures. Reinforced concrete is characterized by its limited ductility, due to its fragile behavior in response to large deformations. The design of concrete structures should carefully spell out the ductility mechanisms in the different elements in a more demanding manner than in the case of steel. The following are the basic criteria for this purpose:

- *Confinement*. The confinement of concrete guarantees preservation of the material under the alternating stress that occurs during earthquakes and, accordingly, permits the development of larger inelastic deformations than are possible in a structure in which the concrete deteriorates.

- *Controlling shear failure*. Shear failure seriously compromises the integrity of any element of reinforced concrete. For this reason design codes usually require a shear design that guarantees that shear resistance is greater than flexure resistance. This is achieved by utilizing as a shear design a value that at the very least corresponds to the plastic yielding from flexure at the end connections.
- *Controlling the reduction in available ductility due to axial load.* Axial compression load drastically reduces the displacement ductility available in a concrete element subject to this load. The phenomenon, which is generally more severe in columns than in structural walls, is usually due to the fact that with heavier compression loads the working stress of the steel is reduced, which can occur with working stress values smaller than yield stresses, which implies an inadequate use of steel for the purpose of developing large inelastic deformations and dissipating energy in this manner. However, it is not always possible to design the sections of columns so that there are heavy traction stresses on the steel, for architectural and economic reasons.

Park and Paulay (*9*) explain at length the behavior and design of reinforced concrete subject to seismic vibrations.

Duration of the earthquake

The effect of the duration of an earthquake on structural behavior has traditionally been ignored in design codes. This is due, in part, to the fact that the accelerations spectrum is insensitive to the duration of the earthquake, since it embodies information only with reference to the maximum response acceleration at some point during the earthquake and disregards what happens afterwards. However, in long earthquakes complex phenomena of degradation of stiffness and resistance can occur, due to the great number of load cycles that the structural elements must endure. Due to this, the design should be different for short and long earthquakes, regardless of the design acceleration.

According to several studies conducted in different countries, the duration of an earthquake correlates increasingly with its magnitude and the distance from the epicenter. In contrast, the ground acceleration decreases with this distance. Thus, there can be earthquakes of equal peak acceleration, which would produce the same design accelerations spectrum but large differences in duration, which would produce harmful effects that would not be detected by this spectrum. As a result, the design of hospitals should take into account seismological information on magnitudes and epicentral distances from the probable sources of energy release that can affect them, so that if there are sources of high probable magnitudes located at great epicentral distances, much longer and possibly more destructive earthquakes than nearby ones can be expected. The 1985 earthquake in Mexico is an example not only of ground amplification effects but also of the effects of long duration, due to the high magnitude (8.1) and distance from the epicenter (350 km).

Isolation and control of vibrations

In recent years there has been a marked increase in the use of techniques of foundation isolation and vibration control in structures located in seismic areas as alternatives to energy dissipation by means of tolerance of damage when structural elements enter the non-linear range. Thus, these systems will undoubtedly be very important in the construction of buildings in general, due to the increasing requirements of structural and non-structural safety in the face of strong earthquakes and the mounting demands for comfort amid environmental vibrations. They can be classified into two groups: systems of isolation and systems of vibration control.

These ***systems of isolation*** are systems that absorb the energy in the excitation base by means of large deformations, damping, or a combination of the two. They can be classified into the following subgroups:

Systems of isolation in the foundation of the building

Among the most common devices utilized to isolate the foundations of buildings are:

- *Rubber and steel supports*. These supports possess great stiffness and resistance to vertical loads, which enables them to resist the loads derived from the weight and use of the structure, while their great flexibility amid horizontal movements enables most of the energy of the earthquake to be dissipated in them. In some cases, the rubber also has high damping characteristics, while in others the support has a lead core that plays the role of damper, which makes combination with dampers unnecessary.
- *Dampers in the foundation*. They can be viscous friction dampers or flexible steel bars anchored in viscous dampers, which are placed in the foundation of the building to reduce seismic energy. In many cases they are sufficient to dissipate the environmental vibrational

energy and the energy of low-intensity earthquakes, but in the case of strong earthquakes they should be combined with steel and rubber dampers. Their damping coefficient can exceed 30%.

Floor isolation system for equipment

Floor isolation systems are devices whose purpose is to isolate the vibrations of floors on which there is electronic or precision equipment that may be damaged or decalibrated by dynamic movement. They are placed between the slab and the frame that supports the floor of the equipment. They consist of rubber or ball-bearing supports and/or viscous dampers to control horizontal vibration, and air springs for vertical vibration.

Vibration control systems have been designed mainly to dampen wind, environmental, and seismic vibrations inside buildings. They can be classified into the following groups:

- *Passive Control Systems*

  Passive control systems are suitable for environmental vibrations, but in the case of earthquakes they are not generally useful except for those of moderate intensity. They include the following mechanisms:

  - *Passive structural dampers*. These dampers of different materials (flexible shear steel, silicone rubber, plastic rubber, bituminous rubber, etc.) are usually placed under the floor diaphragms to help absorb the induced energy uniformly throughout the entire height of the building.
  - *Lever dampers*. They consist of dampers that convert the horizontal movement of the structure into a broad vertical movement of double vertical dampers placed on the ends of one lever, whose viscosity reduces the vibrations.
  - *Passive resonant pendulums*. This system controls vibrations by placing a massive pendulum, weighing close to 1% of the total mass of the building, on the roof; the pendulum is designed with a period equal to that of the structure so that it begins resonating. This results in the maximum efficiency of the pendulum, because owing to its zero stiffness it induces an inertial force opposite to the elastic forces of the structure at its highest point. A force is thus obtained that at least partly counteracts the inertial forces derived from the weight of the structure. Among the systems used is the simple pendulum with masses of steel or concrete, multiple pendulums, or water tanks with spill control. In the

latter case the tank can be used as a normal reserve water tank in the building.

- *Systems of Active Control*

  The systems of active control differ from the previous ones in that they gear the working conditions to excitation in accordance with the reading of sensors placed in various parts of the building, the signals from which are read and analyzed by computer. They normally have alternate sources of electric power so as to avoid the consequences of a cutoff in the event of strong earthquakes. The following types are worthy of mention:

  - *Active pendulums*. These are pendulums like the ones described above; they incorporate an active force opposite to the inertial forces of the structure in every cycle, a force that the automatic control system calculates by computer from the signals provided by the sensors.
  - *Active structural controllers*. They consist of steel diagonals connected to a mechanism that receives signals from the computer and the sensors. The purpose of the mechanism is to modify the stiffness of the diagonal in accordance with the signals so as to remove the structure from the area of resonance in each cycle.

CHAPTER 3

STRUCTURAL AND NONSTRUCTURAL VULNERABILITY OF HOSPITALS

PROBLEMS OF CONFIGURATION

By their nature hospital buildings tend to be large and complex which means that in many cases their configuration is complex. By configuration we will not refer here just to the form of the structure in the abstract, but to the close interrelation of the type, layout, fragmentation, resistance, and geometry of the structure of the building, an interrelation from which derive certain problems of structural response to earthquakes. The planning of a hospital must take into account the fact that one of the greatest causes of damage to buildings has been improper architectural-structural configurations. It can be said in general that a departure from simple structural forms and layouts is punished severely by earthquakes. In addition, unfortunately, the usual methods of seismic analysis do not succeed in quantifying most of these problems appropriately. In any case, given the erratic nature of earthquakes, as well as the possibility that the design level will be exceeded, it is advisable to avoid proposing hazardous configurations, regardless of the degree of sophistication that may be achieved in the analysis of each case.

This chapter discusses briefly the impact of configuration on the seismic response of buildings, as well as the corrective mechanisms. It should be emphasized that due to its complexity and to its close relationship with the proposed size and shape of the construction, the problems of configuration should be addressed primarily in the stage when the spatial layout of the building is preliminarily defined and throughout the formal and structural design stage. For this reason it is a subject that architectural designers must understand in its entirety (*10*).

Problems of plane configuration

The problems discussed below are pertinent to the layout of the structure in the horizontal plane, with regard to the form and distribution of the architectural space.

Length

The plane length of a structure influences its structural response in a way that is not easy to determine by the usual methods of analysis. Since the movement of the ground consists of a transmission of waves, which occurs with a velocity that depends on the mass and stiffness characteristics of the supporting soil, the excitation that takes place at one point of support of the building at a given moment differs from the excitation at another, the difference being greater to the extent that the length of the building is greater in the direction of the waves. Figure 12 illustrates this problem graphically for short and long buildings and for waves of horizontal and vertical movement. It can be seen that short buildings adjust more easily to the waves and receive a similar excitation at all supports, unlike long buildings. This fact seemingly brings a reduction in the response accelerations of the structure, due to the smaller excitations at some supports. However, certain undesirable stresses that do not occur in short buildings appear: compression and tensile stresses in the diaphragms and foundations due to the horizontal waves, and shear stresses due to the vertical waves, which affect the entire structure.

Secondly, long buildings are also more sensitive to the torsional components of ground movements, because the differences in the transverse and longitudinal movements of the supporting ground, on which this rotation depends, are greater. In quantitative terms, the length of the building should not be judged as short or long in absolute terms but only in relation to the dynamic characteristics of the soil.

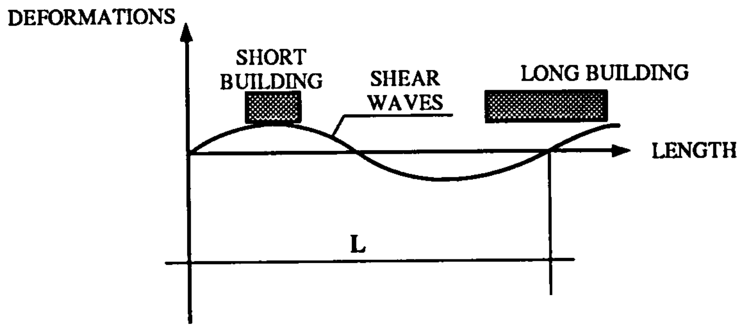

FIGURE 12. INFLUENCE OF LENGTH ON STRUCTURAL RESPONSE

The parameter:

$$\tau = \frac{L}{v}$$

where L is the length of the building in the direction of analysis and v is the speed of the waves in the ground (which can be taken as the speed of the shear waves for the horizontal movements and as the speed of the compression waves for the vertical ones), can be used as criterion for measurement. It can be said that the building is considered long for values greater than 0.2 secs.

The usual solution to this problem of excessive building length is to partition the structure in blocks by inserting joints in such a way that each block can be considered short. These joints should be designed so that they permit an appropriate movement of each block without danger of pounding, as described later.

Flexibility

The flexibility of a structure under seismic loads can be defined as its susceptibility to suffer large lateral deformations between the different floors, known as drift. The leading causes are the distance between the support elements (clear spaces or clearances), their vertical clearance, and their stiffness. Depending on its degree, flexibility can result in:

- Damage to the non-structural elements attached to contiguous levels.
- Instability of the flexible floor or floors or of the building in general.

Both problems have been examined in the previous chapter (see Fig. 5).

Lack of redundancy

As was described in chapter 2, the seismic-resistant design of structures includes the possibility of damage to elements during the most intense earthquakes. From this point of view, the design of the structure should see to it that resistance to seismic forces does not depend largely or totally on a small number of elements, because their failure could result in partial or total collapse right after the earthquake, due to the weakness of the remaining ones. It should be borne in mind that the usual seismic-resistant design evaluates certain design forces corresponding to very intense earthquakes and with a response of the structure at the inelastic level, forces that it distributes among the elements in accordance with their stiffness, that is, according to an elastic model. Disregard of this contradiction has frequently been the cause of errors in the structural layout of buildings, since at the time of a strong earthquake the structural elements will respond in accordance with the inelastic pattern, that is, according to resistance, and will behave differently from the elastic pattern of strict distribution in accordance with stiffness, because the entire structure enters the non-linear range. Therefore, if most of the resistance of the structure is concentrated in a single element (for example, staircase walls and elevators), and as can be expected in accordance with the design, this element fails, the structure is no longer able to absorb the seismic energy in the time following the earthquake. In this regard, the goal should be to distribute the resistance to seismic forces among as many elements as possible, regardless of the elastic stiffness provided. Figure 13 illustrates this principle.

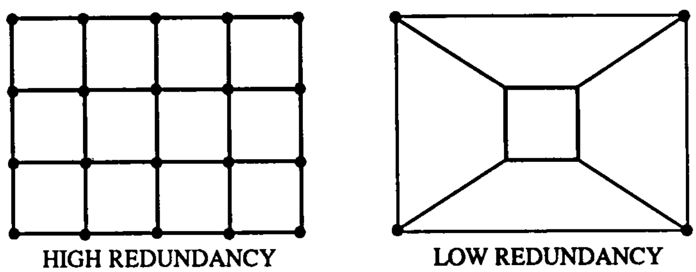

FIGURE 13. REDUNDANCY

Logically, the problem of a lack of redundancy is usually linked to the problem of flexibility, because fewer elements in a given area entail the presence of large gaps between the supports and, accordingly, less lateral stiffness of the structure.

Torsion

Torsion has been the cause of major damage to buildings subject to strong earthquakes, damage that ranges from the sometimes visible warping of the structure (and accordingly its loss of image and reliability) to structural collapse. For the purposes of this section we will consider three types of torsion problems: elastic torsion, inelastic or accidental torsion, and natural torsion.

Elastic torsion. As mentioned in chapter 2, torsion, in elastic terms, takes place because of the eccentricity of the center of mass in relation to stiffness. The three major cases that can give rise to this situation in the plane are:

- The positioning of the most rigid structure asymmetrically with respect to the gravity center of the floor.
- The placement of large masses asymmetrically with respect to stiffness.
- A combination of the two previous situations.

It should be kept in mind that partition and front walls that are attached to the vertical structure are usually very rigid and, accordingly, at least while their resistance is greater than the demand of the earthquake, participate structurally in the response to it and can be causes of torsion, as in the common case of corner buildings.

If height is also considered, the situation with respect to torsion can become even more complicated when there are vertical irregularities, such as the setbacks. Indeed, the upper part of the building transmits an eccentric shear to the lower part, which causes downward torsion of the transition level, regardless of the structural symmetry or asymmetry of the upper and lower floors.

Quantitatively, an eccentricity between mass and stiffness can be considered large when it exceeds 10% of the plane dimension under analysis. In a case like this, corrective measures should be taken in the structural layout of the building.

Inelastic or accidental torsion. As mentioned in the previous section, it should be kept in mind that the effective response of the structure to an intense earthquake occurs, and in a very complex way, more in terms of resistance than stiffness, and thus multiple factors, such as the type of design, the quality of the construction of the elements, the differences in their manufacture, any damage that occurs etc., determine the actual response. For this reason, there may be torsions on the various floors in addition to the assumed torsion, owing to the elastic model, which does not encompass variables other than the stiffness of the elements. Consequently, the design should include in the elastic model an additional torsion that is usually called accidental torsion, although it is preferable to consider it more as a foreseeable torsion under the principles of non-linear design than as something fortuitous, as the name suggests. Under many design codes, the total design value of the eccentricity between mass and stiffness is taken to be the sum of elastic eccentricity plus an additional eccentricity owing to this factor:

$$e = e_e + e_i$$

Here, the first term on the right represents elastic eccentricity, which is calculated implicitly in the spatial matrix analysis described in chapter 2, and the second is inelastic eccentricity, which is usually taken to be a fraction of the plane dimension of the building perpendicular to the direction under analysis. This fraction fluctuates between 5 and 10 percent.

Natural torsion. By its very nature, an earthquake involves rotational components that induce torsional movement of the structure from its foundation. As a result, the phenomenon of torsion should be considered even in symmetrical buildings. Normally, this natural torsion is envisioned in design codes within the calculations of inelastic torsion. However, for certain buildings (long ones, resting on soft ground, etc.) a more detailed analysis may be desirable. Newmark and Rosenblueth (*11*) illustrate the way to estimate the torsion spectrum in accordance with the principal parameters of the design earthquake.

As with all problems of configuration, the problem of torsion should be addressed starting in the stage of size and form design. The necessary corrections to the problem of torsion can be summarized in the following points:

- Natural and accidental torsions should be considered unavoidable due to the nature of the phenomenon and to the nature of inelastic structural design. For this reason, the suggestion is to provide buildings with so-called perimetric stiffness, which seeks to brace the structure against any possibility of rotation and to distribute torsional resistance among several elements, in accordance with the need for redundancy.
- For the purposes of controlling elastic torsion, careful attention should be paid to the layout of the structure in plane and in

elevation, as well as to the presence of and need for isolation of the partition walls that could be structurally involved at the time of an earthquake, as will be discussed later on. In all this the objective should be the greatest possible symmetry between stiffness and mass.

Flexibility of the diaphragm

Chapter 2 mentioned the difference between rigid and flexible behavior of the floor diaphragm. Figure 1 illustrates clearly that in the second case there are larger lateral deformations, which are in principle detrimental to the non-structural elements attached to contiguous levels. Second, the steel reinforcement of the vertical structure by the diaphragm is done deficiently, for which reason there is more stress on some elements and less on others, in comparison to calculations under the rigid diaphragm hypothesis.

There are several reasons why this type of flexural stress may occur. Among them are the following:

- *Flexibility of the diaphragm material.* Among the usual building materials wood presents the biggest drawbacks from this point of view.
- *Aspect ratio of the diaphragm.* Since this is flexural stress, the larger the length/width ratio of the diaphragm, or of a segment of it, the greater its lateral deformations may be. In general, diaphragms with aspect ratios greater than 5 can be considered flexible.
- *Stiffness of the vertical structure.* The flexibility of the diaphragm should also be judged in accordance with the plane distribution of the stiffness of the vertical structure. In the extreme case of a long diaphragm in which all elements have equal stiffness, better performance of the diaphragm can be expected than in the case in which there are major differences on this subject.
- *Openings in the diaphragm.* Large openings in the diaphragm for purposes of lighting, ventilation, and visual connections between floors cause flexible areas to appear within the diaphragm, which impede the rigid steel reinforcement of the vertical structures.

There are many solutions to the problem of flexibility of the diaphragm and they depend on its cause. In principle, for important structures such as hospitals, building floors with flexible materials such as wood should be avoided. Second, as with length effects, buildings that have a large plan aspect ratio should be segmented by joints. With respect to the third cause, very large differences in stiffness between the elements of the vertical structure should be avoided. Finally, large openings in the diaphragm should be studied carefully, in order to provide for stiffening or, if this is not possible, the segmentation of the building into blocks.

Concentration of stresses in the plane

This problem arises in buildings consisting of complex planes and is very common in hospital buildings. The definition of such a plane is one in which the line joining any two sufficiently distant points of the plane lies largely outside the plane. This occurs when the plane is composed of wings of significant size oriented in different directions (H, U, L shapes, etc.). In them, every wing can be likened to a cantilever built into the remaining body of the building, a site at which it would suffer smaller lateral deformations than in the rest of the wing (figure 14). For this reason large stresses appear in the transition area, which frequently cause damage to non-structural elements, to the vertical structure, and even to the diaphragm.

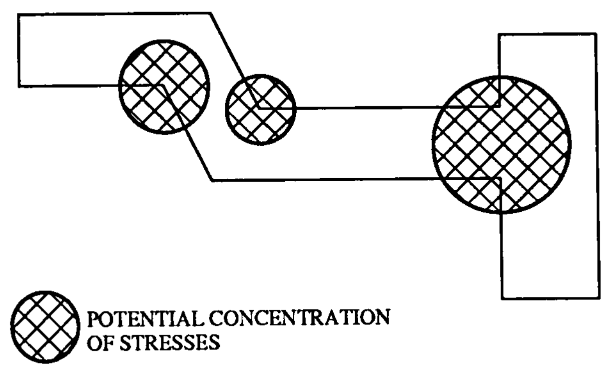

FIGURE 14. CONCENTRATION OF PLANE STRESSES

A detailed study of this concentration of stresses requires a three-dimensional analysis that includes the flexibility of the diaphragm in the model. However, as in previous cases, it is preferable to opt for practical solutions instead of sophisticated analyses. In this case the solution ordinarily adopted is to install seismic separation joints, such as the ones mentioned in the case of long buildings. These joints enable each block to move on its own without being tied to the rest of the building, thus breaking the pattern of cantilever effect on each wing. The joints must obviously be wide enough to allow the movement of each block without hammering. To this end, the minimum width s of the joint can be calculated in accordance with the formula:

$$s = \delta_A + \delta_B$$

where the terms on the right represent the inelastic deformations, measured with respect to the foundation that are expected for blocks A and B next to the joint. In addition, for joints in areas of circulation, flexible connection mechanisms that guarantee normal transit through the joint without constituting a rigid bond between the blocks should be provided.

Problems of vertical configuration

Concentrations of mass

This problem is caused by high concentrations of the total mass of the building on a given floor because heavy elements, such as equipment, tanks, storerooms, files, etc. have been placed there. The problem worsens the higher up this heavy floor is, since the seismic response accelerations also increase upward, and thus there is a larger seismic response force at that point and a greater possibility of overturning.

From the inelastic standpoint, concentrations of mass on a floor lead to a greater absorption of energy on that floor than on the remaining floors, as shown in figure 11. It can be seen that almost all of the inelastic energy is absorbed at the upper level, and as a result the accumulated ductility demand factor becomes so high that it is difficult or impossible to achieve by the ordinary means of earthquake-resistent ductile design. It is also clear that the ductility factor of the lower floors is smaller than required in the design, thus making it obvious that the dissipation of energy is irregular in elevation due to the concentration of mass and that only a part of the structure is responsible for most of the inelastic energy; as a result, there can be heavy damage to that section.

The architectural design of these buildings should place whatever areas represent unusual weights in basements or in isolated structures near the main body of the building. In cases in which for topographical reasons large amounts of water must be stored at high elevations, independent towers should be preferred for this purpose, instead of towers attached to the main building. Otherwise, a careful study should be made of the effects of mass concentration on energy dissipation.

Weak columns

The seismic design of frames, which considers the inelastic response of the structure, tries to ensure that the damage produced by intense earthquakes occurs to beams, not columns, since there is a greater risk of building collapse from the latter type of damage. However, many buildings designed according to earthquake-resistance codes have failed for this reason. These failures can be grouped into two categories: 1) columns with less resistance than beams; 2) short columns.

In the first case, the frame has been designed in such a way that the resistance provided to the beams that meet at a connection is greater than that of the respective columns. When the connection is twisted by seismic movement, the columns yield before the beams, and as a result plastic joints form in the columns before they do in the beams, and collapse may occur because of instability. Some design codes include the provision that the total flexural resistance of columns that meet at a connection must be at least 6/5 of the resistance of the beams that meet there, in every direction. This mechanism insures that plastic joints form in the beams before they do in the columns.

The second case involves columns that are too short in relation to the dimensions of their section:

$$l/h \leq 2$$

where l is the free unsupported length of the column and h is the dimension of the section in the direction under analysis. There are several reasons why the value of the free length is reduced drastically:

- Lateral confinement of the column by dividing walls, front walls, retaining walls, etc.
- Placement of slabs at intermediate levels.
- Location of the building on a slope.

Short columns are the cause of serious failures in buildings under seismic excitations. Their failure mechanism is fragile because it occurs from shearing rather than from flexure. Indeed, failure from flexure takes place for a shearing force equivalent to:

$$Q_u = 2M_u/l$$

For a very short column the value of this equivalent resistance is very high in comparison with the resistance to the direct shear, which is why the failure takes place because of the latter.

Since this is a delicate problem, the effective length of the columns should be studied carefully in the construction design. The most appropriate solutions in the case of all kinds of walls that impede the free movement of the column consist basically of placing the wall in a different plane from that of the column, or in separating the wall from the column by means of joints. In the case of buildings with intermediate levels, the architectural design should consider locating the columns outside the transition line between the levels. Finally, on slopes, the foundations of the columns ought to be sunk at greater depths.

Soft stories

Several types of architectural and structural schemes lead to the formation of so-called "soft stories," that is, stories that are more vulnerable to seismic damage than others, since they have less stiffness, less resistance, or both. The usual characteristics are:

- Greater elevation of the story
- Interruption of vertical structural elements on the story
- Construction on slopes.

The first case frequently arises because of a desire to place greater masses at certain levels of the structure, usually for technical reasons (equipment requirements, etc.) or aesthetic reasons (the building's appearance at the access levels, etc.). As a result, stiffness on the floors in question weakens due to the greater elevation of vertical elements, and resistance weakens due to a reduction in the equivalent flexural shear resistance Q_U of the columns, as mentioned above, and to the increased importance of second-order or stability effects.

The interruption of vertical elements of the structure has proven to be the cause of multiple partial or total collapses in buildings subject to earthquakes. The reason is that the floor on which the elements are interrupted has greater flexibility than the others, thus aggravating the problem of stability. Mainly, however, a sudden change of stiffness takes place, causing a greater accumulation of energy on the weaker story, as was stated in the previous chapter. The most common cases of such interruption, which usually occurs for size, form, or aesthetic reasons, are the following:

- Interruption of the columns
- Interruption of structural walls (shear walls)
- Interruption of partition walls, conceived erroneously as non-structural, aligned with frames

On this point refer again to what is said about the application of the energy method in figure 11.

Setbacks

Setbacks in the volume of a building are generally used because of city-planning requirements relating to illumination, proportion, etc. However, from the seismic perspective, they cause sudden changes in stiffness and mass and accordingly entail the corresponding problems mentioned above of concentrating the damaging energy in the floors near the area of the sudden change. In general, the aim should be to make the transitions as smooth as possible in order to avoid such concentration.

Inverted setbacks should be avoided in seismic areas, since they also involve a serious risk of overturning, as mentioned with respect to the distribution of mass.

The previous applications of the energy method confirm the fact that in a seismic-resistant design it is not enough to provide resistance to some inertial forces without relation to the distribution of mass, stiffness, and resistance. Dynamic analysis meets this need in part, since it takes into account the distribution of mass and stiffness, which makes it better than the simplified method. But because it is an elastic method, it leaves out the distribution of resistance and, on the other hand, like the simplified method, it assumes a uniform distribution of ductility demand, which is far from true in the irregular cases. For these reasons, the application of elastic methods should be examined carefully when the aim is to dissipate energy by means of permanent deformations of the structure.

NON-STRUCTURAL ELEMENTS

The design of every structure subject to seismic movements should consider that the non-structural elements in the building, such as ceilings, panels, windows, and doors, as well as equipment, mechanical and sanitary installations, etc., must withstand the movements of the structure. Moreover, it should be borne in mind that the excitation of the non-structural elements, caused by these movements of the structure, is in general greater than excitation at the foundation, and thus it can be said that the safety of the non-structural elements is more compromised in many cases that the safety of the structure itself.

Despite the foregoing, the seismic design of structures usually gives little importance to these elements, so much so that many design codes do not include standards in this regard. Experience with recent earthquakes shows in many cases excellent behavior of the structure designed in accordance with modern criteria of seismic-resistance, accompanied unfortunately by a poor response of the non-structural elements. However, if the safety of the occupants of a building and of people endangered by the collapse of such elements is taken into account, as the cost of replacing these elements, and the losses involved in interrupting the operations of the building itself, the importance of seismic design of the non-structural elements within the general layout of the building can be understood.

In the case of hospitals, the problem is of major importance, for the following reasons:

- Hospital facilities should be kept as intact as possible in the event of a strong earthquake, due to their importance in responding to a seismic disaster in the city or region in which they operate. This goes for both structural and non-structural elements.
- At the time of an earthquake, hospitals house a large number of patients who are practically incapable of evacuating the building, in contrast to the occupants of any other building. This means that the failure of non-structural elements should not be tolerated in this type of structure, as it indeed tends to be in others.
- Hospitals have a complex network of electric, mechanical, and sanitary facilities, as well as large amounts of usually expensive equipment, all of which are indispensable for the normal operation of a hospital, as well as for responding to an emergency. As a result, hospitals cannot allow the movement of the structure to bring about failures in these facilities and equipment, which in turn would cause the functional collapse of the building.
- The ratio of the cost of non-structural elements to the total cost of the building is much higher in hospitals than in other buildings. Indeed, whereas in apartment and office buildings it is approximately 60%, in hospitals, due mainly to the cost of medical equipment and special facilities, it reaches between 85% and 90%.

Performance analysis

A detailed analysis of the response of the non-structural elements and of the attachments to the structure is usually conducted by one of the following methods:

- Dynamic analysis of the structure/attached components combination.
- Analysis of the response of the component based on a dynamic analysis of the response in time of the level on which it is located.
- Analysis of the response of the component based on a dynamic analysis of the maximum (spectral) response of the level on which it is located.

The first type of analysis is justified in the case of major attachments to the structure, in terms of their weight and size, such as chimneys, tanks, etc. For the remaining elements the third type of analysis may be sufficient. Based on the design force of a story, as obtained by dynamic analysis of maximum response or by simplified analysis, the acceleration of the story can be obtained:

$$a_j = \frac{f_j}{m_j}$$

which can be considered as the acceleration at the base of the component. Obviously, since forces increase with height, the higher the placement of components the more unfavorable. In addition, it is necessary to remember that non-structural components are subject to total acceleration, which is the response acceleration of the story relative to the ground plus the acceleration of the ground. Accordingly, for the design of a non-structural component the following formula can be used:

$$a_{ne} = a_j + \max \ddot{u}_s$$

and therefore, the amplification of acceleration with respect to the foundation of the building would be given by:

$$M_x = \frac{a_j + \max \ddot{u}_s}{\max \ddot{u}_s}$$

Since the design of the element is frequently the responsibility of professionals who are unfamiliar with the management of seismic variables, ATC-3 Standards recommend the use of the equation:

$$M_x = 1 + \frac{h_x}{h_n}$$

where:

h_x = height from the foundation of the building to level x

h_n = *total height of the building*

Moreover, it is necessary to consider the flexibility of the element-support system itself. The components attached to a given level can be considered as belonging to two categories, rigid and flexible. In the first case, the component will respond with the same acceleration of the story on which it is anchored, while in the second its response will be different and, usually, greater than that of the story, depending on the flexibility of the support and of the component itself. This is the case with equipment installed on resilient bases to isolate the vibration and sound produced by their operation. If it is assumed that the acceleration of the story is sinusoidal, the additional amplification of the seismic force in the element, in accordance with the theory of harmonic vibrations, is given by:

$$M_e = \sqrt{\frac{1+[2\xi_e\frac{T_e}{T}]^2}{[1-(\frac{T_e}{T})^2]^2+[2\xi_e\frac{T_e}{T}]^2}}$$

where the subindex *e* denotes the dynamic characteristics of the element. The most critical condition occurs when the period of the element and the period of the structure are similar. For a period ratio of 1 and an equipment damping of 2%, there would be an amplification factor of 25. Several factors suggest that the use of this equation yields values that are too high in comparison with observations in actual earthquakes and that for design purposes a smaller amplification should be considered. Among these factors are: (a) the consideration of the ductility of the support system of the element; (b) the variation of the period of the structure as it enters the inelastic range; and (c) the chaotic nature of the seismic excitation waves. For these reasons, the ATC-3 Standards (*12*) recommend a minimum value of:

$$M_e = 2, \ if\ 0.6 \le \frac{T_e}{T} \le 1.4$$

and of 1 for the other cases.

In calculating the seismic design forces of the non-structural elements it is necessary to consider, finally, the importance of the non-structural element and the importance of the structure, in order to establish a performance criterion appropriate to both. The required performance criteria are classified into three groups, to each of which corresponds a performance index ***P***:

PERFORMANCE CRITERION		
Designation	**Level of performance**	**P**
S	Superior	1.5
G	Good	1.0
L	Poor	0.5

In accordance with this, the design force of the component is reduced to the following equation in the formulation of the ATC-3 Standards:

$$f_e = A_v\ C_e\ P\ M_e\ M_x\ W_e$$

where:

A_v = *effective story acceleration*

C_e = *design seismic coefficient of element*

The tables containing the specifications of the seismic coefficient for architectural and mechanical components can be found in the ATC-3 Standards. Similar formulations for calculating the forces in non-structural elements are found in other design codes.

Interaction with the structure

In addition, the non-structural elements attached to two successive diaphragms, such as partition walls and panels, windows, doors, etc., should be designed in order to support the drift of the story on which they are located. This requirement for maximum deformation differs from the requirement for the design force mentioned above, in that the former regulates the design of the element for its own inertial forces and the latter refers to the displacement induced by the movement of the diaphragms to which it is tied (*13*).

This requirement is of great importance due to the usually fragile materials used in the construction of the elements in question, such as masonry, asbestos, glass, etc. Design codes usually require limiting story drift for the purpose of indirectly ensuring protection for non-structural elements attached to the diaphragms. The limit that the ATC-3 code accepts for hospitals is 0.01 times the free height of the floor for the design earthquake. It can be assumed that compliance with this limit affords satisfactory indirect protection to high-quality non-structural materials and construction. However, if there are doubts in this regard, it is a good idea to provide isolation systems for such elements so that they do not undergo these deformations.

In the case of windows, for example, the high fragility of glass makes the use of suitable isolation almost compulsory. As for masonry walls joined to the structure, isolation should be considered with regard to the overall concept of the structure design. If these walls are not seen as part of the seismic resistance system and if they in turn may cause torsion problems due to their asymmetrical position, or problems of soft stories due to their concentration on just a few stories, then it is appropriate to isolate them. Rosenblueth (*6*) presents several models for isolating walls from the diaphragm and the frame.

In the opposite case, that is to say, when the walls do not cause problems because of their arrangement in plane and elevation, it is appropriate to consider them as part of the seismic-resistant structure. This is important since the seismic response of the structure as a whole may be very different from the response reported by the model that disregards the presence of the walls. Indeed, the variation of stiffness in the model leads to different design forces, both in moderate and intense earthquakes. Moreover, the vertical loads on the columns are very different from one model to another.

Due to the poor adherence of walls to the frame and because the latter deforms mainly in the flexural mode and the former in the shearing mode, there is a separation between the two in the areas of tension, and a narrowing in the areas of compression. For this reason, in the interaction between the two only one band of the wall works in compression.

The international literature has proposed several ways of determining the width of the compression band in the wall, which can determine the area of the diagonal element that should be included in the analysis.

The usual modes of failure of a wall are sliding and compression. The force along the diagonal under which the sliding starts can be evaluated as:

$$R_s = \left(0.9 + 0.3\frac{l}{h} \right) f_{bs}\, h\, t$$

where:

$f_{bs} \equiv$ *resistance of adherence between the mortar and the masonry*

$t \equiv$ *thickness of the wall*

The force along the diagonal that is needed to produce the compression failure of the wall can be calculated as:

$$R_c = \frac{2}{3}\, t\, \alpha\, f_m'\, \sec\theta$$

with:

$$\alpha = \frac{\pi}{2}\left(\frac{4\, E\, I\, h_m}{E_m t \sin 2\theta}\right)^{1/4}$$

In principle it is desirable to prevent sliding failures since they lead to the formation of short columns in the frame. Accordingly, the cementing mortar of the masonry must be of the highest quality possible. To avoid failure from compression, wall retaining meshes should be placed in the finish of the wall; they, in turn, can be utilized as a device to increase the ductility of the structure.

Isolation

The decision to isolate the masonry of the structure should be made carefully, since the masonry must be anchored appropriately in order to compensate for its independence from the structure and to prevent its collapse, which, in the case of hospitals, can be catastrophic. It is usually advisable to isolate the masonry of the structure in the following cases:

- Whenever its plane position tends to cause strong eccentricities of stiffness and, thus, severe torsion.
- Whenever it tends to produce excessive stiffness of one or several stories in relation to the rest, which in that case would become relatively weak.

CHAPTER 4

EVALUATION AND REDUCTION OF VULNERABILITY

VULNERABILITY ANALYSIS

The previous chapters have dealt with the aspects that should be considered in the analysis and design of new hospital buildings, in accordance with recent theories on seismic resistance. However, doubts arise concerning the safety of existing hospitals, mainly when they are needed to respond to a seismic emergency and have been designed only to handle gravity loads. In these cases it becomes imperative to conduct the most detailed review possible of the structure's capacity to withstand moderate and strong earthquakes. It must be kept in mind that the difficulty of building new hospital facilities in seismic areas, due to high costs, makes retrofitting the existing ones imperative. The retrofitting design necessarily entails an analysis of the available capacity of resistance and ductility during earthquakes, as well as of the functional, organizational, and administrative vulnerability of the hospital, before it is remodeled.

This chapter discusses the principal methods for conducting this structural review. It should be pointed out, in addition, that this review will be insufficient if not accompanied by a detailed review of the non-structural elements, the collapse or failure of which may cause serious problems of functional vulnerability in essential buildings.

The international literature offers various methods of conducting a detailed seismic vulnerability analysis of a structure; a long list of these methods can be found in the bibliography. In general, the methods can be classified into the following groups:

- Qualitative methods
- Experimental methods
- Analytical methods

The first are methods designed to evaluate rapidly and simply a group of diverse buildings and to select those that warrant more detailed analysis. The main use of these methods is the large scale evaluation of buildings for the purpose of quantifying seismic risk over a broad region of a city. Their findings, however, cannot really be taken as conclusive in any particular case (*14*). Some of these methods constitute the first level of evaluation of the analytical methods, as in the case of the "Japanese method" and the evaluation designed by Iglesias (*15*) for Mexico City. The ATC-21 method is also worth mentioning(*16*). These are primarily qualitative methods, in which the structure receives a grade based on such considerations as its condition, irregularity in plane and elevation, its relation to the ground, etc. Such grading in general does not require office calculations. The first level of the Japanese method, in contrast, requires the computation of certain variables, and its equations are closely related to those of the higher levels.

For the post-seismic recovery of essential buildings a more careful analysis is desirable, for which purpose analytical and experimental methods are available. The latter determine the dynamic behavior of the structure by a direct measurement of environmental vibrations. Thus, they have the disadvantage of providing information only on the dynamic characteristics of the structure under vibrations of low amplitude, making them inadequate for easing concerns about resistance, dissipation of energy, etc. For this reason they must be complemented by purely analytical methods.

Purely analytical methods are generally used to evaluate in detail the possible vulnerability of a structure during earthquakes of different orders of magnitude. In general, all non-linear and hysteretic analysis of a structure subject to ground movements constitute a vulnerability analysis. However, the applicability of these methods is debatable for several reasons:

- The great complexity of the model, which justifies its use only in very special but at the same time very simple cases, conditions that seldom coincide.
- The need to conduct the analysis using several types of earthquake measurement, in order to cover the various possible impacts on the structure, which complicates the situation.

For these reasons, various analytical methods that are closer to the usual design practices have been developed. Noteworthy among them are the "Japanese," the "American," and the "energy" methods.

Japanese method

This method (*17*, *18*), issued officially in Japan by the Ministry of Construction for reviewing concrete buildings constructed in seismic areas, has three levels of evaluation, which run from simple to detailed, and is, in general, supremely rigorous. It is based on an assessment of the seismic behavior of each floor of the structure by means of an index that incorporates the following aspects:

- Resistance of the vertical elements (columns and walls), C.
- Their capacity of ductility, F.
- The condition of the structure and its performance in previous earthquakes, T.
- Influence of the form, asymmetry in plane, concentrations of mass and stiffness, openings in the diaphragm, etc., S_d.
- Influence of topographical and geotechnical conditions, G.

The first two indices are combined in a maximum of three groups per floor, through the index:

$$E=C.F$$

and, in turn, the indices E of a floor are averaged probabilistically in order to obtain a single index E per floor, by the method of the square root of the sum of the squares. The groups in question show ductile behavior and are formed on the basis of the characteristic ductility factor, which is obtained in the manner explained later.

For each group the indices T and S_d are then obtained, and finally index I_s, which has as a value:

$$I_s=E.S_d.G.T$$

Values of I_s greater than a reference value I_{so} ensure, under this method, an appropriate behavior of the story.

The value of I_{so} is calculated as:

$$I_o = E_s . Z . G . U$$

where:

E_s = the basic value of seismic behavior, related only to the method of evaluation

Z = the seismic zone factor, considered in relation to the probable hazard in the area, with maximum value of 1.0

U = the importance factor of the structure for recovery after an earthquake.

The method has three levels of evaluation, the first of which is commented on above. The application of the second one requires knowledge of the steel reinforcement of the vertical structural elements, and the third requires knowledge of the beams as well. The most important feature of this method is that it attaches greater significance to resistance analysis than to an analysis of the internal stresses that a given earthquake could cause in the elements of the structure. It can be said that the application of this method does not, in general, require a detailed analysis of these internal stresses. In addition, the method attaches great importance to establishing the mechanisms of failure and energy dissipation of the elements, which are classified into various groups in accordance with their behavior and type of failure. This analysis starts with an evaluation of the ductility capacity factor ***m*** of every element, produced by:

$$m = m_o - k_1 - k_2$$

$$m_o = 10 . (\frac{V_n}{Q_n} - 1)$$

where:

V_n = nominal shear resistance of the vertical supports. In the case of columns, the calculation includes the axial load of the element.

Q_n = shear resistance provided by the plastic yielding of the ends of the column, that is,

$$Q_n = 2\frac{M_n}{L}$$

M_n being the nominal resistance to flexure of the element and ***L*** its length.

k_1, k_2 = factors relating to the possible buckling of the reinforcement bars and the level of shear stress.

Based on the calculation of Q_n, the possible mode of failure of every element can easily be established, so that values of $Q_n < V_n$ yield flexural failures and, in contrast, shearing failures, that is, fragility failures.

Once this evaluation has been done, each type of element is subdivided into a maximum of three groups in accordance with values representative of ***m***, and the respective indices ***C*** and ***F*** are obtained, which gives the common index ***E*** that describes both the resistance and the ductility available in the structure.

American methods

ATC-14 and ATC-22, (*19*, *20*, *21*), endorsed by the Federal Emergency Management Agency of the United States, are methods that propose a simplified review of the structure, using approximate equations to estimate stresses and deformations. Method ATC-14 was developed in connection with the SEAOC design concept of working stress, while ATC-22 is related to the ATC-3 concept of limit design. In both cases, the review seeks to obtain demand/capacity ratios (in contrast with ATC-14) for the structural elements subject to the various stresses.

Unlike the previous method, this attaches great importance to the analysis of internal stresses and to the resistance of the elements to these stresses, since its final objective is to evaluate the ratios seismic force demand/redundant capacity of resistance to the force, D_e/C_e:

$$\frac{D_e}{C_e} = \frac{Q_e}{(Q_n - Q_v)}$$

where Q_e = internal force caused only by the seismic load
Q_n = resistance of the element to the force
Q_v = internal force caused by the vertical load.

These values should be arranged in descending order so that the higher ones represent the elements that are at greatest risk of failure.

However, the method has the serious defect of not providing the tools to evaluate clearly the ductility capacity of the structure in its different types of elements and on its different floors, as indeed the previous method does. Instead, it classifies buildings according to an overall indicator of ductility as a function of their structural system, based on a table that is a bit more detailed than the ATC-3 code. Recent experience with strong earthquakes and the theoretical "energy" methods reveal, however, substantial differences in the ductility demand between the different floors of a structure, and even between different elements, as will be shown later, and for this reason the overall assessment of the energy absorption capacity in the entire building by means of a single index is clearly insufficient.

Energy method

The energy method (*8*), mentioned in chapter 2 of this document, offers an approach that is radically different from the usual one for both designing and reviewing buildings subject to seismic movements. For the purpose of reviewing existing structures it has the virtue of clearly identifying the soft stories of the structure, the elements of the floors that will tend to fail first, and the ductility demands associated with the energy absorbed on each floor, and accordingly it appropriately reflects the probable condition of the building in the event of a strong earthquake.

REDUCING VULNERABILITY

As was seen previously, evaluating the condition of an existing structure can raise serious doubts as to its capacity to withstand seismic events (*22*). Some countries have undertaken campaigns to reinforce existing buildings to reduce their vulnerability prior to the occurrence of an event. In principle, it could be thought that such a reduction should be compulsory for essential buildings that handle emergencies arising from earthquakes and that are deemed inadequate in one or several respects by the evaluations mentioned.

Common problems

The causes of weakness in a structure can be ascertained through the diagnosis conducted in accordance with the methods outlined above. Following is a list of some of such causes (*23*):

- *Low overall capacity to dissipate energy*. This problem is common in buildings constructed in accordance with design standards that do not provide for resistance to earthquakes. Large separations of stirrups in beams and columns are observed in them, as well as little compression strengthening in beams in the areas close to the connections, and the absence of connection confinement.
- *Little resistance over what is required to handle gravity loads*. This is logical in the case of designs intended only to handle

such loads, at least with respect to beams. In the case of columns, the situation may be more or less critical due to the complexity of the moment-axial force interaction.

- *Errors in the structural model.* Errors and inconsistencies of various kinds can be found in the original concept of the structure subject to vertical loads. Among them could be a disregard for moments of flexure in the design of columns, conceiving beams as simply supported, and constructing them as continuous elements, etc.
- *Deficient stiffness and resistance in one or two directions.* In a design concept exclusively for gravity loads with slabs in one direction, beams are not necessary in the working direction of the slab. For this reason they are frequently omitted. This makes the structure particularly flexible and weak in this direction. In the case of slabs in two directions, on the other hand, the problem can be worse if they have been constructed without beams, since the aforementioned problems occur in both directions and since the slabs are usually not prepared to withstand the shearing stress arising from the earthquake.

In addition to these problems, problems related to configuration and walls are also common.

Retrofitting design

In keeping with the above, the modification of the structure should seek to solve these problems in the following four ways:

- Increase in the overall capacity of energy dissipation
- Increase in resistance
- Reduction of energy concentration in plane and elevation
- Stiffening

The analysis and design of the structural model of the strengthened structure should be carried out in clear consideration of aspects such as the following:

- The impact of stiffness variation on spectral response. In the acceleration spectrum stiffness variation can significantly affect the overall response of the structure.
- The response of the old elements that have not been modified but whose connection to the diaphragm causes them to be involved in the overall response of a story.
- The impact of the isolation of walls on the stiffness of each floor.

- The additional elements that should be constructed when seismic movement joints are created in the diaphragms.
- The interrelationship between the mechanisms of stiffening, increase in resistance, and ductility.
- The change of stresses in the ground and the foundation.
- The relationship between the construction system and building maintenance.
- The cost of the modification.
- The architectural, functional, and aesthetic aspects of retrofitting.

The usual systems of structure reinforcement tend to incorporate the following additional elements (*24*). (See also annex 2.)

- *Exterior building walls.* This solution is commonly used when limitations of space and continuity of building use make work in the periphery preferable. To ensure that stresses are transmitted by the diaphragm to the walls, collector beams are used on the edges of the slab. It is not recommended for very long buildings.
- *Buttresses.* Unlike the previous elements, they are placed perpendicular to the face of the building. In addition to providing stiffness, they are useful in taking the overturning moment in tall buildings. Due to space limitations they are not always feasible.
- *Interior building walls.* When work is possible inside the building, they are an alternative that must be considered in long buildings, in which the flexibility of the diaphragm should be reduced. They are usually inserted through perforations in the diaphragms, through which the reinforcement bars pass.
- *Frame walls.* Both inside and outside buildings, one practical solution to the problem of stiffness and resistance is to fill frame openings with concrete or reinforced masonry walls. Due to the connection with the column, the stresses on them will change substantially. If the strengthening of the column is sufficient for the new situation, the connection with the wall may be done solely by means of soldered bolts. Otherwise, a sheathing of the column monolithic with the wall should be constructed.
- *Braced frames.* Another common solution is to include several steel frames with diagonals strongly anchored to the diaphragms, as a substitute for stiff walls. Also, only the diagonals joined to existing frames can be constructed when the frames demonstrate resistance to the stresses placed on them by the new system, especially, the axial forces in the columns and the shearing forces in the connections.

- *Sheathing of columns and beams*. Used for frame systems, this system is commonly used on most of the columns and beams in a building, for the purpose of increasing their stiffness, resistance, and ductility alike. In most cases these systems differ basically in the way in which the new covering is joined to the existing column.
- *Construction of a new frame system*. On occasions total retrofitting is possible by attaching the old structure to new external perimetric frames. This is usually combined with the incorporation of internal structural walls perpendicular to the longitudinal direction of the frames.

Coordination of retrofitting

Modifying the seismic vulnerability of a hospital building is usually more complex than on any other type of building. There are several considerations that make this type of work different in health-care facilities.

- Normally the building cannot be vacated to carry out the retrofitting, particularly when the structural modification is being done as a preventive measure before a likely earthquake.
- The scheduling of the work should take into account the operation of the different medical services, so as not to seriously disrupt the functioning of the hospital or unjustifiably shut down certain types of services.
- It must be anticipated that there will be a large number of unforeseen tasks due to the difficulty of identifying precisely certain details of the construction process prior to the start of the work.
- Likewise, it should be realized that the non-structural elements are complex and that it will be difficult to identify changes in or effects on the architectural finishes prior to the start of the structural modification.

In view of the above, retrofitting should be undertaken in accordance with a very detailed work program that covers considerations relative to the operation of services at each stage of the process. In the same way, there should be proper coordination with administrative, medical treatment, and hospital maintenance personnel.

The cost of reducing a hospital's vulnerability cannot be ascertained unless a detailed design of the structural solution and of its implications for the non-structural elements is done. However, this situation should not prevent the formulation of a somewhat precise schedule that requires minimal adjustments as work proceeds.

The costs of retroffiting are usually relatively high if it is done quickly. However, if the work is done in stages, resources can be used more gradually and within the range of expenditures for hospital maintenance.

The high economic and social benefits of improving the structural performance of vulnerable hospital buildings has been demonstrated in every instance. Although the cost of retrofitting may be considered high at times, it will always be insignificant in comparison to the service budget or the cost of repair or physical replacement. The following pertinent questions could be posed in each case, for example: How many scanners would be equivalent to the cost of the retrofitting? How many scanners does the hospital have? The answers may be surprising, without even considering all of the other components, equipment, and assets that are generally found in the building. This calculation does not, of course, take into account the human lives that are directly or indirectly affected or the social cost that the loss of the service represents.

CHAPTER 5

UNIVERSITY AND PROFESSIONAL TRAINING

ADAPTATION OF CURRICULA

In Latin America, structural engineering is, in general, a compulsory subject in civil engineering schools. However, the aspects related to the analysis and design of structures subject to seismic loads have not been widely addressed at the graduate level, much less in basic university education. In general, it could be said that seismic-resistant design is a subject has not been sufficiently treated, except for some isolated efforts by the most prestigious schools of engineering in each country, which offer specialization or continuing education courses in the subject.

Consequently, the seismic-resistant structural design of health facilities such as hospitals has been practically ignored, not only because it is not dealt with in the courses or subjects that specialize in seismic-resistant analysis and structural design, but also because buildings in the health sector are usually treated as conventional structures or because attention is paid only to minor differences that do not lead to their being considered buildings that require special treatment.

As with other essential structures, hospital design deserves to be treated at the undergraduate and graduate level. In the first case, in order to make professionals aware of how important the operations of this type of facility are, and in the second, in order to assess the level of acceptable risk and to determine the most appropriate analysis and design requirements in accordance with the analyses of costs, safety, and operation that each case requires.

Therefore, at the undergraduate level the recommendation is to include in structure analysis and design courses sections on the importance of good performance by hospital buildings, particularly in the courses on the application of construction code requirements. Moreover, at the graduate level the recommendation is to address the subject of the minimum acceptable level of safety for such facilities as hospitals and to apply techniques and methodologies such as the ones described in this document. An educational strategy like the one recommended will not yield major results in the short term, but it will contribute to a conceptual change and a change of attitudes among future professionals.

Some resistance to the inclusion of new subjects in the curriculum of engineering schools is common, and as a result the relevance and far-reaching importance of the subject in professional practice must be demonstrated. Previous experience indicates that the process usually begins with the particular interest of a certain teacher or as a result of conferences or seminars that stimulate general interest. In this regard, individual events can play a fundamental role, such as a recent earthquake, which can be the catalyst for the start of these activities within the university.

It is important to mention the close relationship that engineers working with structural design and hydraulic, electrical, gas, etc. facilities ought to have with architectural designers, which is the reason why the subject should also be multidisciplinary. As was mentioned before, once the need for treatment of the subject has been made explicit, it is less difficult to incorporate it into curricula. For this reason, engineers, who are usually more aware of the problem, should underscore the need to attach due importance to the subject within other disciplines. In the medium term this will create favorable conditions so that aspects relating to disaster mitigation in hospitals can be gradually incorporated into curricula.

CONTINUING EDUCATION

Since formal education does not offer tangible results in the short term, a strategy needs to be developed to bring knowledge to practicing professionals, be they staff members in the health sector, consultants, or educators.

Given that the most effective strategy for incorporating the subject into the curricula of engineering schools is to promote training and continuing education, acquainting professional and union associations and undergraduate and graduate students to the subject is an indispensable step before promoting curriculum adaptation.

Short courses in continuing education and lectures on the performance of hospitals under seismic loads at congresses, symposia, seminars, and workshops on structures, seismic engineering, reinforced concrete construction, etc., can awaken the interest of professionals involved in hospital construction and in many cases inspire them to begin giving due consideration to risk mitigation in existing health facilities and in designing new buildings.

Professional and union associations and universities can be of great help in making this process of professional training serious and pertinent by expanding the coverage that can be achieved within institutions. This educational technique is an excellent means of bringing together experiences and proposing alternatives to formal education.

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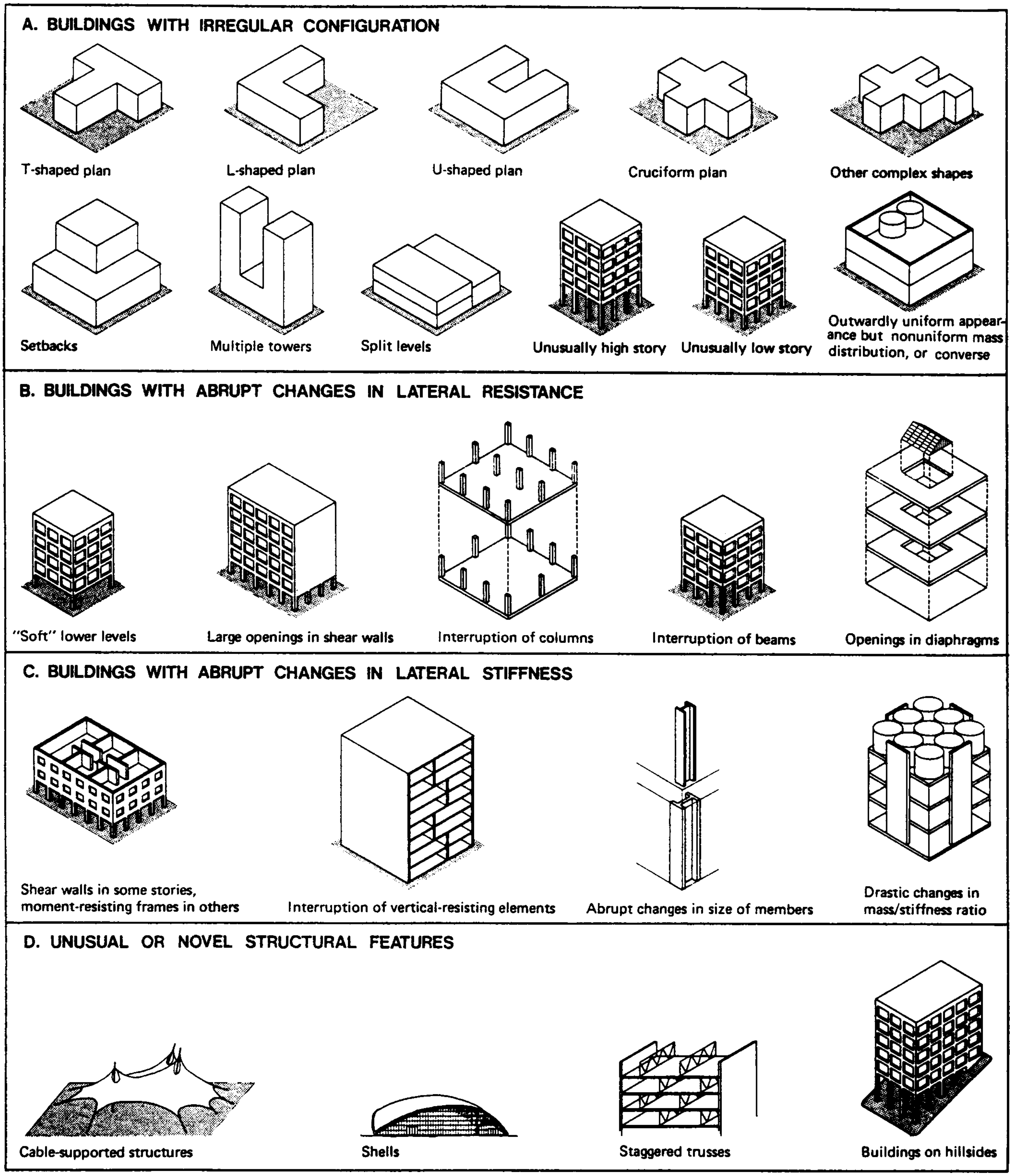
ANNEX 1: CONFIGURATION CONSIDERATIONS

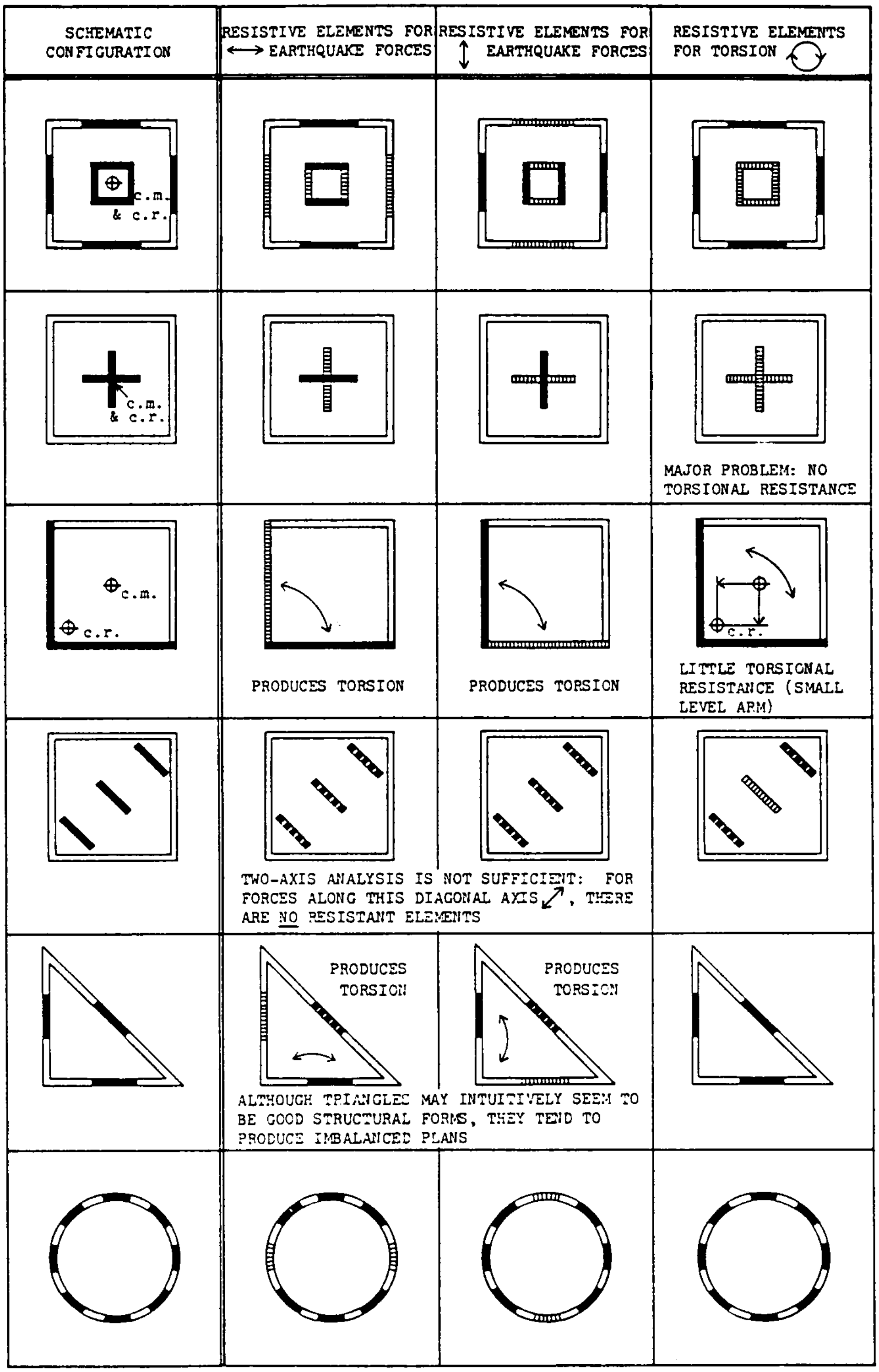
- *IRREGULARITIES IN STRUCTURES*
- *LOCATION OF SHEAR WALLS*

IRREGULARITIES IN STRUCTURES



Graphic interpretation of "irregularities in structures or in systems of frameworks," from *SEAOC Recommended Lateral Force Requirements and Commentary*. (Taken from *Configuration and Seismic Design of Buildings*, Christopher Arnold and Robert Reitherman, John Wiley and Sons, New York, 1982, p. 8. Reprinted with permission.)

LOCATION OF SHEAR WALLS



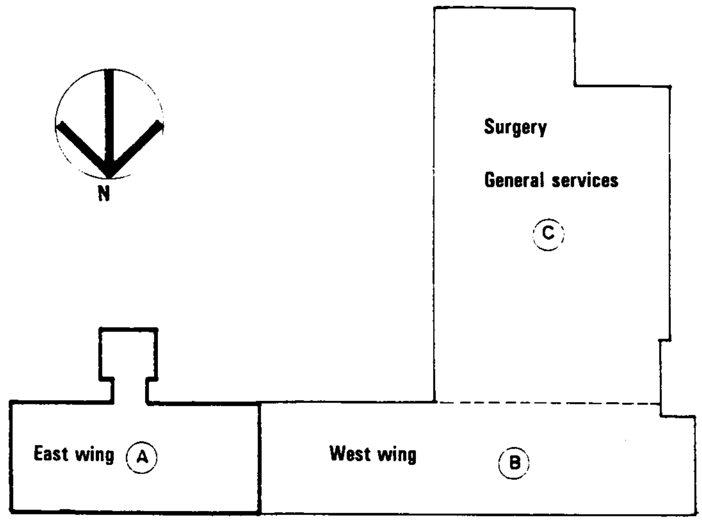
Location of shear walls. Schematic plans can be seen as sets of resistant elements with various orientations (to resist translation) and with variable distances to the center of rigidity (to resist rotation or torsion). (Taken from *Configuration and Seismic Design of Buildings*, Christopher Arnold and Robert Reitherman, John Wiley and Sons, New York, 1982, p. 42. Reproduced with permission.)

ANNEX 2:
EXAMPLES OF REHABILITATION

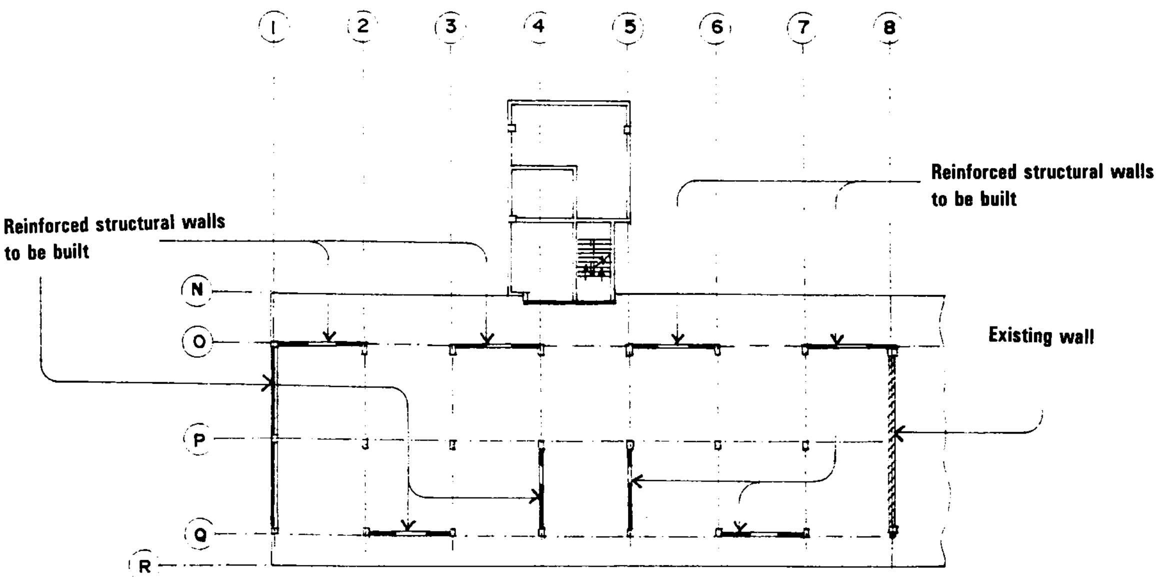
- *HOSPITAL NACIONAL DE NIÑOS*
- *HOSPITAL MEXICO*
- *HOSPITAL MONSEÑOR SANABRIA*

Taken from *"Comportamiento de Hospitales en Costa Rica durante los Sismos de 1990"*, Miguel F. Cruz A., PAHO, 1991.

HOSPITAL NACIONAL DE NIÑOS

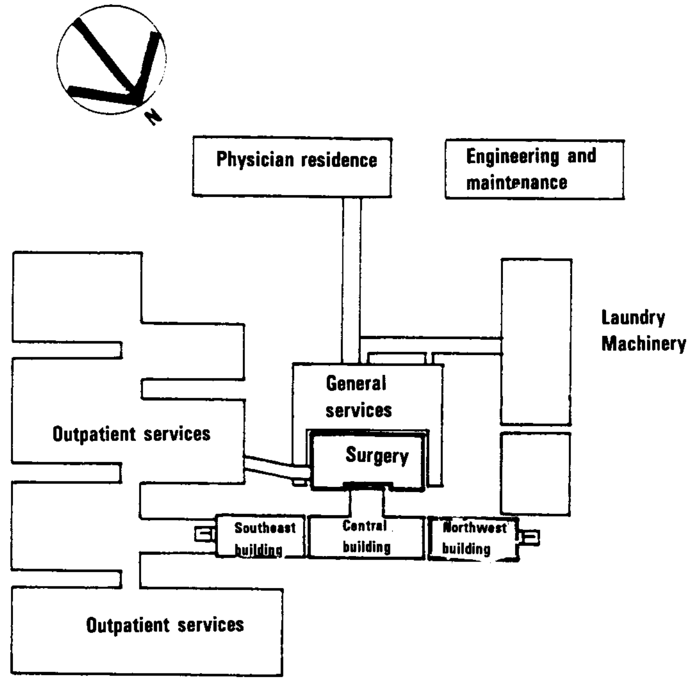


GENERAL LAYOUT--HOSPITAL NACIONAL DE NINOS



ARCHITECTURAL PLANE FOR RETROFITTED EAST WING
(HOSPITAL NACIONAL DE NINOS) (Scale 1:400)

HOSPITAL MEXICO



DISTRIBUTION OF PRINCIPAL BUILDINGS (HOSPITAL MEXICO)

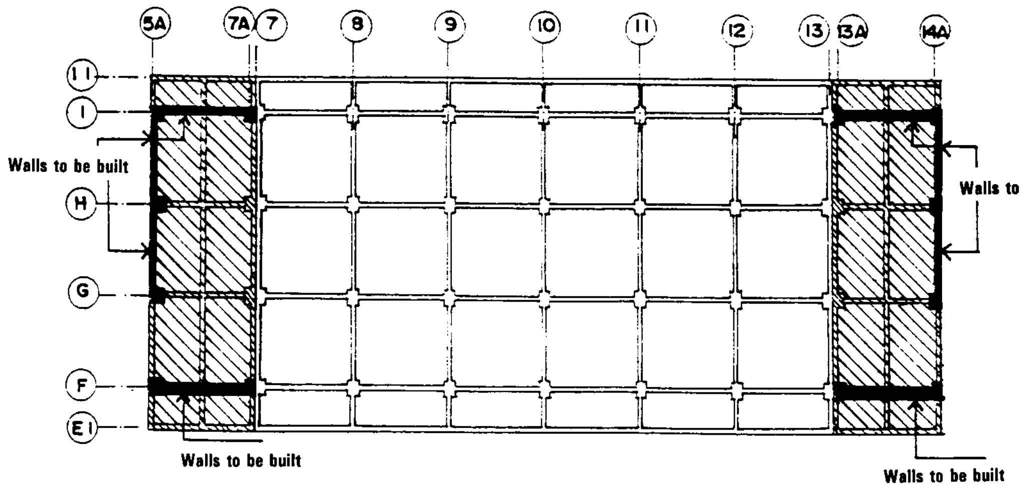
KEY:

Reinforced structural walls to be built

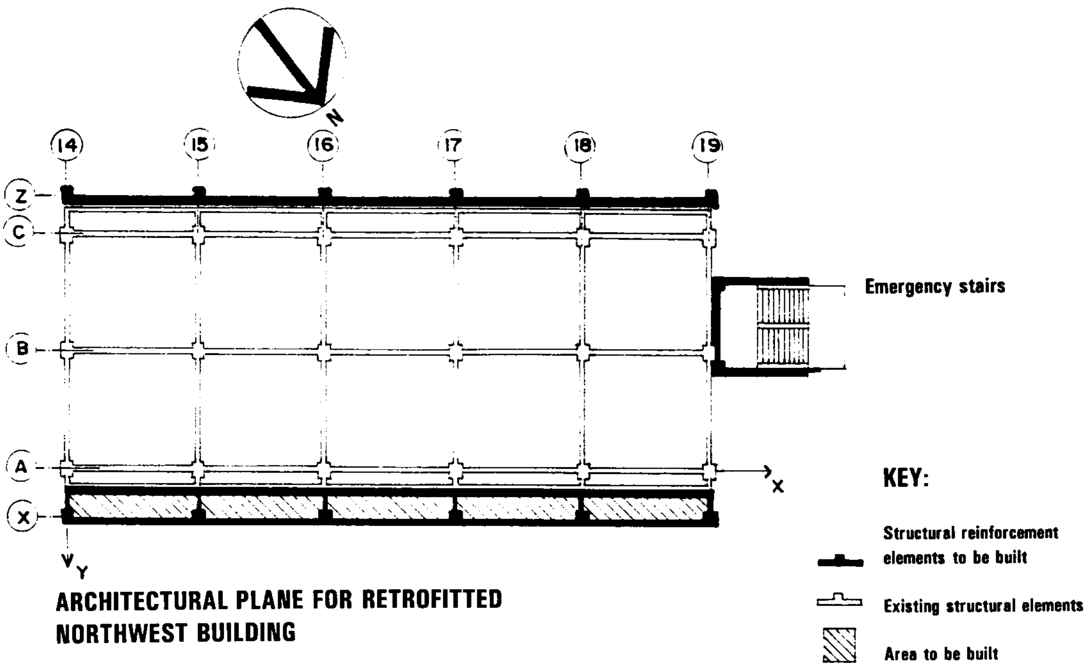
Structural elements to be built

Existing structural elements

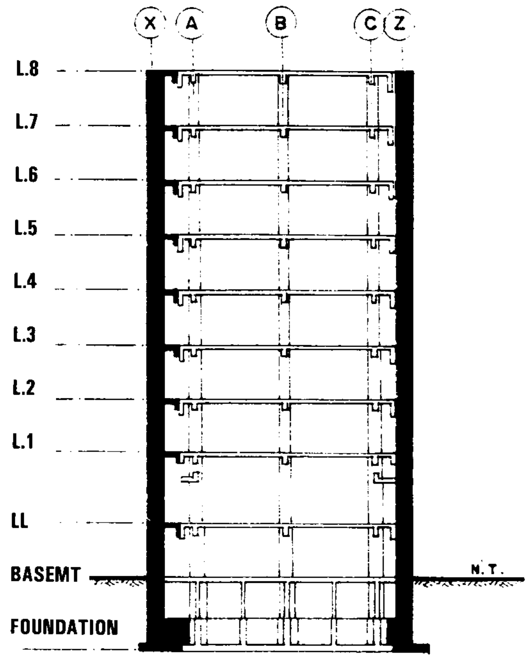
Area to be built



STRUCTURAL PLANE FOR SURGERY BUILDING

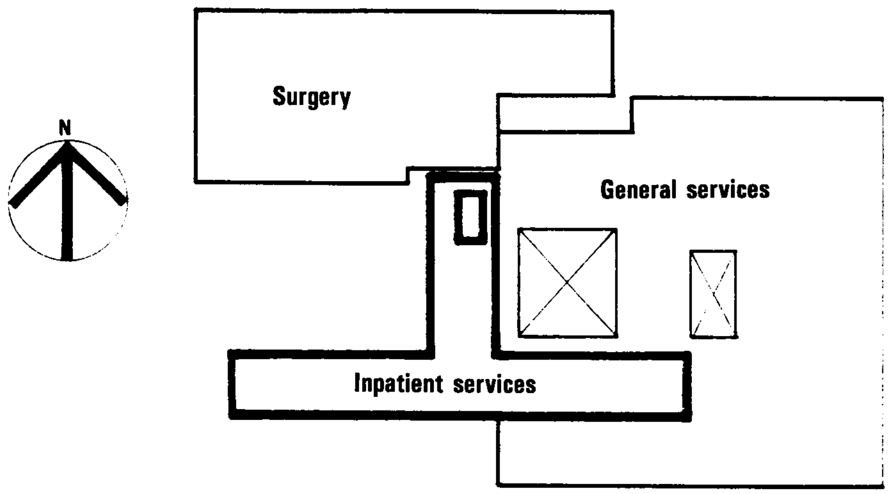


ARCHITECTURAL PLANE FOR RETROFITTED NORTHWEST BUILDING

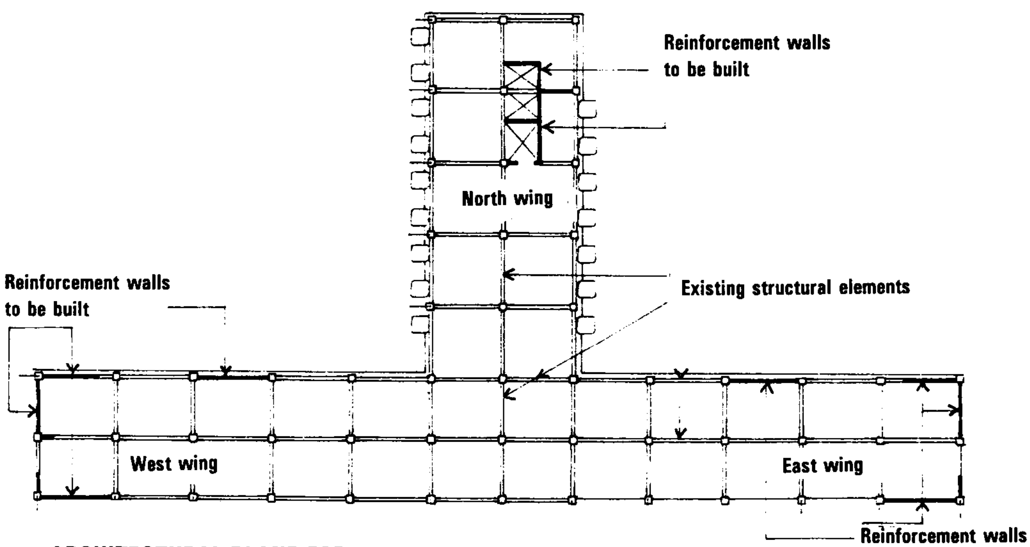


TRANSVERSE SECTION

HOSPITAL MONSEÑOR SANABRIA



GENERAL LAYOUT OF HOSPITAL MONSENOR SANABRIA



ARCHITECTURAL PLANE FOR
RETROFITTED INPATIENT SERVICES (Scale 1:500)

ANNEX 3: RECOMMENDED BIBLIOGRAPHY

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